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CIVIL ENGINEERS.

(INSTITUTED 1852.)

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CONTENTS.

PAPERS.

NO.	MONTH.		PAGE
687	*January.*	Erosion of River Banks.—Discussion on Paper No. 600.	
		By WILLIAM STARLING	1
		ARTHUR HIDER	4
		SAMUEL H. YONGE	12
		HENRY B. RICHARDSON	14
		HORACE M. MARSHALL	20
		WM. P. CRAIGHILL	24
		A. F. WROTNOWSKI	26
		J. A. OCKERSON	26
688	*do.*	Storage and Pondage of Water.	
		JOSEPH P. FRIZELL	29
689	*do.*	Reclamation of the Potomac Flats at Washington, D. C.	
		PETER C. HAINS	55
690	*do.*	Railroad Location.	
		MICHAEL L. LYNCH	81
		Discussion on Paper No. 690.	
		By C. E. MOORE	91
		W. B. STOREY, Jr.	91
		HOWARD G. KELLEY	94
		D. H. AINSWORTH	95
		C. D. PURDON	98
		R. G. TARR	99
		E. H. BECKLER	100
		HENRY GANNETT	102
		J. Q. BARLOW	103
		W. M. DAVIS	105
		KENNETH ALLEN	107
		FOSTER CROWELL	109
		CHARLES H. SNOW	111
		J. E. GREINER	113
		JAMES R. MAXWELL	115
		HARVEY LINTON	116
		BAILEY WILLIS	117
		CHARLES W. STANIFORD	118
		P. F. BRENDLINGER	118
		MICHAEL L. LYNCH	125
691	*February.*	The Water Works of Denver, Colo.	
		JAMES D. SCHUYLER	135
		Discussion on Paper No. 691.	
		By R. C. GEMMELL	163
		A. FTELEY	166
		JAMES OWEN	168
		EDW. P. NORTH	169
		E. E. OLCOTT	170
		L. L. TRIBUS	170
		C. O. GLEIM	171
		J. D. FOUQUET	172
		R. L. HARRIS	172
		J. D. SCHUYLER	173

31418

NO.	MONTH.		PAGE
692	*February.*	Train Loadings for Railroad Bridges.	
		THEODORE COOPER	174
		Discussion on Paper No. 692.	
		By GEORGE H. BLAKELEY	185
		CHARLES S. CHURCHILL	189
		JOHN A. FULTON	192
		EDWIN THACHER	193
		WARD BALDWIN	193
		J. C. BLAND	194
		J. E. GREINER	205
		JOSEPH M. WILSON	207
		HENRY B. SEAMAN	210
		W. B. PARSONS	210
		T. KENNARD THOMSON	211
		DAVID L. BARNES	211
		J. A. S. WADDELL	213
		THEODORE COOPER	219
693	*do.*	Protecting Piles against the *Teredo Navalis* on the Louisville and Nashville Railroad Company's Lines.	
		R. MONTFORT	221
		Discussion on Paper No. 693.	
		By GEO. H. THOMSON	233
		JAMES DUANE	234
		GEORGE HILL	234
		A. McC. PARKER	235
		RICHARD LAMB	238
		JOHN A. McDONALD	240
		H. COMER	240
		H. W. BRINCKERHOFF	241
		F. COLLINGWOOD	241
		CHARLES MACDONALD	242
		L. J. LE CONTE	242
		ERNEST PONTZEN	244
		R. MONTFORT	245
694	*March.*	The Tower of the New City Hall at Philadelphia, Pa.	
		C. R. GRIMM	249
		Discussion on Paper No. 694.	
		By JOSEPH M. WILSON	273
		GEO. HILL	274
		J. A. McDONALD	275
		J. FOSTER FLAGG	275
		C. R. GRIMM	275
695	*do.*	The Removal of a Defective Pivot Pier, and its Reconstruction.	
		HOWARD G. KELLEY	277
696	*do.*	Lining a Water-Works Tunnel with Concrete.	
		DESMOND FITZGERALD	294
		Discussion on Paper No. 696.	
		By R. W. LESLEY	307
		C. B. BRUSH	309
		P. F. BRENDLINGER	311
		JAMES OWEN	314
		W. B. PARSONS	314
		KENNETH ALLEN	315
		O. F. NICHOLS	316
		JOHN BOGART	317
		L. LUIGGI	317
		JOS. P. FRIZELL	319

NO.	MONTH.		PAGE
696	*March.*	Discussion on Paper No. 696.—(*Continued*).	
		FOSTER CROWELL	321
		JAMES DUANE	322
		P. D. FORD	323
		E. SHERMAN GOULD	324
		E. E. R. TRATMAN	325
		J. KENNARD THOMSON	325
		J. J. R. CROES	326
		DESMOND FITZGERALD	326
697	*do.*	Spirals and their Use on Railroads.	
		A. S. C. WÜRTELE	329
698	*do.*	The Electric Station of the Citizens' Light and Power Co., of Rochester, N. Y.	
		ROBERT CARTWRIGHT	335
		Discussion on Paper No. 698.	
		By WM. P. CRAIGHILL	357
		H. W. BRINCKERHOFF	358
		B. R. GREEN	358
		CHAS. E. EMERY	359
		ROBERT CARTWRIGHT	360
699	*do.*	The Renewal of the Channel Pier of the Cincinnati and Muskingum Valley Railway Bridge over the Scioto River.	
		MORTON L. BYERS	361
		Discussion on Paper No. 699.	
		By S. MUNCH KIELLAND	369
		MORTON L. BYERS	370
700	*do.*	Driven Wells of the Plainfield Water Supply System.	
		L. L. TRIBUS	371
		Discussion on Paper No. 700.	
		By P. K. YATES	378
		C. B. BRUSH	380
		A. S. TUTTLE	382
		W. KEIRSTED	383
		H. V. HINCKLEY	385
		JOHN W. HILL	386
		R. WILLARD WARE	388
		L. L. TRIBUS	391
701	*April.*	Cableways.	
		SPENCER MILLER	397
702	*do.*	An Instructive Eye-Bar Test.	
		A. C. CUNNINGHAM	415
		Discussion on Paper No. 702.	
		By PERCIVAL ROBERTS, Jr	417
		JAMES CHRISTIE	418
		R. H. THURSTON	418
		CHARLES MACDONALD	420
		L. L. BUCK	422
		J. M. KNAP	422
		WM. DE H. WASHINGTON	423
		G. H. BLAKELEY	424
		GEORGE S. MORISON	425
		WILLIAM METCALF	426
		G. LEVERICH	431
		A. C. CUNNINGHAM	434
703	*do.*	The Development and Recent Improvement of Concrete-Iron Highway Bridges.	
		FR. VON EMPERGER	438

NO.	MONTH.		PAGE
703	*April.*	Discussion on Paper No. 703.	
		By L. L. Buck	457
		C. T. Purdy	458
		Joseph M. Wilson	459
		Wm. P. Craighill	460
		Foster Crowell	461
		George Hill	461
		W. G. Triest	466
		E. B. Gosling	467
		Carl Gayler	467
		Bernt Berger	469
		W. R. Hutton	472
		Geo. A. Just	473
		J. F. O'Rourke	475
		John Bogart	476
		B. R. Green	477
		Fr. von Emperger	480
704	*May.*	Dredging Operations.—Discussion on Paper No. 689.	
		By Robert A. Cummings	489
		Foster Crowell	491
		J. D. Van Buren	492
		E. B. Gosling	494
		F. S. Washburn	494
		R. S. Buck	495
		R. L. Harris	495
		Chas. B. Brush	496
		Peter C. Hains	497
705	*do.*	The Merchants' Bridge Terminal Railway Viaduct at St. Louis, Mo.	
		Robert Moore	500
706	*do.*	The Relation of Wheels to Frog Points and to Guard Rails.	
		Archibald A. Schenck	509
		Discussion on Paper No. 706.	
		By M. J. Becker	518
		Henry G. Prout	520
		Geo. R. Hardy	520
		H. W. Brinckerhoff	522
		L. L. Buck	522
		A. A. Schenck	522
707	*do.*	The Use of Canvas in Water-Tight Bulkheads.	
		M. Meigs	524
		Discussion on Paper No. 707.	
		By S. Bent Russell	536
		M. Meigs	537
708	*do.*	General Notes on the Great Kanawha Improvement, Tripping Bars and Improved Hurters on Chanoine Wicket Dams, etc.	
		Addison M. Scott	539
		Discussion on Paper No. 708.	
		By Wm. P. Craighill	546
		J. P. Frizell	547
709	*do.*	Storage and Pondage of Water.—Discussion on Paper No. 688.	
		By Wm. E. Worthen	552
		A. Fteley	553
		Chas. B. Brush	553
710	*June.*	Address at the Annual Convention, at the Cataract House, Niagara Falls, N. Y., June 20th, 1894.	
		Wm. P. Craighill	555

NO.	MONTH.		PAGE
711	*June.*	A Concrete Sewer on Piles.	
		EUGENE LENTILHON	569
		Discussion on Paper No. 711.	
		By CHAS. MACDONALD	576
		A. A. STUART	576
		WM. BARCLAY PARSONS	577
		W. W. KENLY	579
		E. B. GOSLING	579
712	*do.*	Failure of a Masonry Pier and a Rock Foundation.	
		WM. BARCLAY PARSONS	580
		Discussion on Paper No. 712.	
		By J. FOSTER FLAGG	583
		BERNARD R. GREEN	583
		JAMES OWEN	584
		P. F. BRENDLINGER	584
		F. COLLINGWOOD	585
		WM. B. PARSONS	586
713	*do.*	The Reconstruction of a Portion of the Substructure of the Johnsonville Bridge.	
		WALTER H. GAHAGAN	587
		Discussion on Paper No. 713.	
		By H. G. KELLEY	597
		M. WARD EASBY	602
		P. F. BRENDLINGER	602
		W. H. GAHAGAN	604

ILLUSTRATIONS.

PLATE.	MONTH.		PAPER	PAGE
I.	*January.*	Map of Lake Providence Reach, Mississippi River	687	5
II.	*do.*	Map of Kempe Bend, Mississippi River	687	20
III.	*do.*	Survey of Potomac Flats, 1792	689	79
IV.	*do.*	" " " 1881	689	79
V.	*do.*	Plan of Potomac Flats Improvement	689	79
VI.	*do.*	Section showing Method of Improvement	689	79
VII.	*do.*	Plans of Tidal Reservoir Outlet	689	79
VIII.	*do.*	Transit and Plane Table	690	117
IX.	*February.*	Profile 30-in. Conduit, Denver Water Works	691	145
X.	*do.*	Temporary Wooden Pipe Arch (plan)	691	147
XI.	*do.*	" " " (photograph)	691	147
XII.	*do.*	Profile 34-in. Conduit, Denver Water Works	691	149
XIII.	*do.*	Buckle Joint, Wooden Pipe	691	151
XIV.	*do.*	Cement Pipe	691	155
XV.	*do.*	Pipe Crossing of Platte River	691	157
XVI.	*do.*	" " " "	691	157
XVII.	*do.*	Filter Tanks, Denver Water Works	691	159
XVIII.	*do.*	Reservoir Covering, Denver Water Works	691	161
XIX.	*do.*	Railroad Reservoir Gate	691	163
XX.	*do.*	Diagram of Equivalent Uniform Loads	692	189
XXI.	*do.*	" " "	692	191
XXII.	*do.*	" " "	692	193
XXIII.	*do.*	Compromise Standard Live Load System	692	213
XXIV.	*do.*	" " " "	692	215
XXV.	*do.*	" " " "	692	217
XXVI.	*do.*	" " " "	692	219

PLATE.	MONTH.		PAPER	PAGE
XXVII.	*February.*	Wrought Iron Shell for Concrete Pile Protection	693	223
XXVIII.	*do.*	Pile Protection by Concrete	693	225
XXIX.	*do.*	" " "	693	227
XXX.	*do.*	Method of Strengthening and Protecting Trestles	693	229
XXXI–XXXIV.	*March.*	Tower of Philadelphia City Hall	694	270
XXXV.	*do.*	Plan of Draw Span, Ouachita River Bridge	695	277
XXXVI.	*do.*	Coffer-Dam and Breakwater, Ouachita River Bridge	695	280
XXXVII.	*do.*	Details of Shoe, Ouachita River Bridge	695	282
XXXVIII.	*do.*	Trestle and Derricks, Ouachita River Bridge	695	285
XXXIX.	*do.*	Pivot Pier, Ouachita River Bridge	695	286
XL–XLII.	*do.*	Concrete Tunnel Lining	696	305
XLIII.	*do.*	View of Scraper at Work	699	364
XLIV.	*do.*	Map of the Water-Shed of the Plainfield Basin.	700	372
XLV.	*do.*	Plan and Detail of Wells, Plainfield Water Supply	700	374
XLVI.	*do.*	Pipe System, Plainfield Water Supply	700	376
XLVII.	*do.*	Water-Pipe Bridge Crossings	700	376
XLVIII.	*April.*	Cableway, Basin Creek Dam	701	407
XLIX.	*do.*	" at Point Pleasant, W. Va	701	409
L.	*do.*	" North Avenue Bridge, Baltimore	701	411
LI.	*do.*	" " " " "	701	411
LII.	*do.*	" Chicago Drainage Canal	701	413
LIII.	*do.*	Aqueduct of Vejus	703	439
LIV.	*do.*	Former Highway Bridge at Moedling	703	441
LV.	*do.*	Present " " "	703	441
LVI.	*do.*	Monier Bridge for Exhibition Purposes	703	443
LVII.	*do.*	Wünsch Bridge at Bolivar, Hungary	703	445
LVIII.	*do.*	Test Arch-Métan System (during test)	703	447
LIX.	*do.*	" " " (broken)	703	449
LX.	*do.*	Comparative Design for Highway Bridges	703	455
LXI.	*do.*	" " " "	703	457
LXII.	*do.*	Diagram for Breaking Loads for Floors	703	487
LXIII.	*May.*	Map of St. Louis Merchants' Bridge Terminal Railway	705	501
LXIV.	*do.*	Profile of St. Louis Merchants' Bridge Terminal Railway	705	503
LXV.	*do.*	Typical Plan of Structure of St. Louis Merchants' Bridge Terminal Railway	705	507
LXVI.	*do.*	Side Elevation of St. Louis Merchants' Bridge Terminal Railway	705	507
LXVII.	*do.*	Floor Plan and Lateral Bracing of St. Louis Merchants' Bridge Terminal Railway	705	507
LXVIII–LXX.	*do.*	Standard Wheel and Rail Sections	706	511
LXXI–LXXIII.	*do.*	Relation of Wheels to Frog Points	706	515
LXXIV–LXXV.	*do.*	Canvas and Plank Bulkheads, U. S. Des Moines Rapids Canal	707	527
LXXVI.	*do.*	Various Uses of Canvas	707	529
LXXVII.	*do.*	Pasqueau Hurters	708	543
LXXVIII.	*do.*	General Section Navigation Pass, Dam No. 7, Kanawha River	708	544
LXXIX–LXXX.	*June.*	Sewer and Overflow Chambers, foot of Canal Street, New York	710	576
LXXXI–LXXXII.	*do.*	Failure of Rock Foundations	712	582
LXXXIII–LXXXIV.	*do.*	Johnsonville Bridge	713	588
LXXXV–LXXXVI.	*do.*	Caisson, Johnsonville Bridge	713	592

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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687.

(Vol. XXXI.—January, 1894.)

EROSION OF RIVER BANKS.—DISCUSSION ON PAPER No. 600.*

By WILLIAM STARLING, ARTHUR HIDER, SAMUEL H. YONGE, HENRY B. RICHARDSON, HORACE M. MARSHALL, W. P. CRAIGHILL, A. F. WROTNOWSKI and J. A. OCKERSON.

WILLIAM STARLING, M. Am. Soc. C. E. (by letter).—This paper is the first important contribution that has been made to the Society on the subject of the regimen of the Mississippi River by the numerous able military and civil engineers connected with the Mississippi service. Surveys and improvements have been going on ever since 1879, a vast amount of interesting work has been done, and a collection of hydraulic and physical data accumulated, which are of inestimable value, and which, in the magnitude of the scale, in number and in precision, are probably unequaled. They are mostly contained, however, in official reports, in the dry and repulsive form of "Government publications," and are practically inaccessible to most of the profession. Only two or three years ago, the library of the Society did not contain a set of the reports of the Mississippi River Commission. The data, moreover,

* "Erosion of River Banks on the Mississippi and Missouri Rivers," by J. A. Ockerson, M. Am. Soc. C. E., *Transactions*, Vol. XXVIII, page 396.

exist only in the rough, and are of comparatively little service, except to those who have kept up with the progress of the work from the beginning. To classify and digest them properly is a task of both skill and labor. No person is better qualified for such a duty than Mr. Ockerson, who has been connected with the survey ever since its organization, has been in charge of most of the work, both in the field and in the office, and is personally familiar with the phenomena he describes. The information which is conveyed to the reader in the convenient form of "tables" represents an amount of labor which can only be appreciated by those who have themselves undertaken work of a similar kind.

Mr. Ockerson's investigations evidently lead him to the conclusion that the Mississippi has never departed very far from its present bed, so far as is shown by any evidence now accessible. This conclusion is not uncontroverted. Many geologists have maintained that the river has wandered entirely over its alluvial basin, and has built it up, indeed, by such excursions as is, I believe, conceded to have been the case with the Missouri. On this supposition, the whole of the flood-plain, from hill to hill, is of strictly alluvial and modern formation, the work of the river in its present state. The sounder doctrine, however, appears to be that of Professor Hilgard, supported by later investigators,* that the floor or foundation of the flood-plain is of pre-alluvial, though still of quaternary, origin, and that it is merely overlaid by a thin "veneer" of alluvium. There is a trough in the middle of the plain through which the river runs, and from side to side of which it wanders, cutting its way on one bank and building on the other. The greater portion of the plain adjoining this central trough is probably of very late origin—in fact, some of it has been built within the memory of man.

Mr. Ockerson's conclusion, that "the lateral oscillation of the stream is really confined to rather narrow limits," is confirmatory of Prof. Hilgard's view.

The statement that "no great amount of caving occurs at high water" appears to me to require some modification. I believe it to be true that on the whole there is more caving on a falling than on a rising river, but I should hesitate to say that the period of greatest activity

* See, especially, Professor McGee's monograph on the Lafayette Formation, Washington, 1891.

was from a medium stage down. I should rather think it was from a high stage down to medium. Instances of high-water caving are so numerous and of such magnitude that they can hardly be regarded as mere exceptions. As a notable example of a cave at the extreme height of a considerable flood, in May, 1886, at Dennis Landing, in Mississippi, a "pocket" caved away to the depth of 500 ft. in one day. At Greenville, in 1891, the bank caved back 1 100 ft., nearly all of which damage was accomplished during extreme high water, while the river was several feet over its banks. Moreover, in this bend, the principal caving is always at this stage. At Shipland Landing, the upper portion of the bend caves during low water, the lower portion at extreme flood. At Leota, the caving is usually at high water, though it is worse when the river is receding than when it is advancing. At this point, for two or three years, the annual rate of caving was 700 or 800 ft. At Longwood, it was nearly the same. At Hall's Landing, in the spring of 1886, the bank caved back about 1 000 ft. In all these instances, the caving ceased at a medium stage. In fact, where the current is very swift, the angle of presentation of the bank very abrupt and the material light and friable, I do not think it makes much difference whether the river is high or low, rising or falling. Where the current is only moderately strong and the other conditions not very unfavorable, the bank may be gradually undermined for a long time, and yet be sustained by the lateral pressure of the water. When the latter falls, its support is withdrawn and the bank falls with it.

Other things being equal, it would seem that the caving should be greatest when the river is up and the volume of water most considerable; and, within my limited sphere of observation, this appears to be the case.

"Low-water caves," I think, are generally immediately due to the shifting of the channel. Very commonly there is a high-water and a low-water channel, one being next, we will say, the left bank, the other next the right. During flood, then, the river will cave on the Mississippi side, while at low water it invades the Louisiana shore. These low-water caves are sometimes very formidable. Duvall's Landing, in Mississippi, is a case in point. This bank caves only between medium and low water, the high-water channel being on the Louisiana side. At flood tide it is almost dead water next the Mississippi bank. Yet from 1887 to 1893 this bank has caved nearly 3 000 ft.

In all caving, the material which has sloughed away lodges at the foot of the bluff bank, forming a talus like that at the foot of a cliff. Here it remains until it is washed away by the current, the caving, in the meantime, of course, being suspended. As soon as the talus is removed, the caving process recommences. This may be repeated several times in one season or not, according to the nature of the situation, the material and the strength of the current. Sometimes the sloughed mass of earth, roots and trunks lies and serves as a natural revetment for years.

Low-water caving may sometimes be due to the local development of an extraordinarily steep low-water slope. For instance, from Arkansas City to Vicksburg, the high-water slope of the water-surface is not far from uniform, while the low-water slope is very variable. For the year 1888, the difference was as follows :

	High-water slope.	Low-water slope.
Arkansas City to Greenville, 40.2 miles...	0.323	0.169
Greenville to Lake Providence, 63.8 miles.	0.327	0.293
Lake Providence to Vicksburg, 56.9 miles.	0.307	0.509

The great development of low-water caving between Lake Providence and Vicksburg may not improbably be due to this cause.

One peculiar form of caving is worthy of notice because it is important in its effects upon works of river improvement. This, so far as I know, was first brought to notice in print by Mr. Ockerson himself in one of his reports to the Mississippi River Commission, in 1884. It consists in a sloughing of the bank due to the immediate neighborhood of ponds or streams. The water from these sources infiltrates the soil of the bank and causes a slipping or sliding of the latter, not usually of great extent, unless other circumstances conspire, yet sufficient to be very destructive to bank-revetment or similar works. This, of course, generally occurs at stages below the medium.

Mr. Ockerson's paper must always be of great value to the student of the Mississippi or similar streams.

Arthur Hider, M. Am. Soc. C. E. (by letter).—The data given in the paper of Mr. Ockerson will be of great value as a basis for future comparison of the changes that are continually in progress in these two great rivers.

The facts presented clearly demonstrate that, while the oscillation of the thalweg of the Mississippi River is very considerable in extent,

PLATE I.
TRANS. AM. SOC. CIV. ENGRS.
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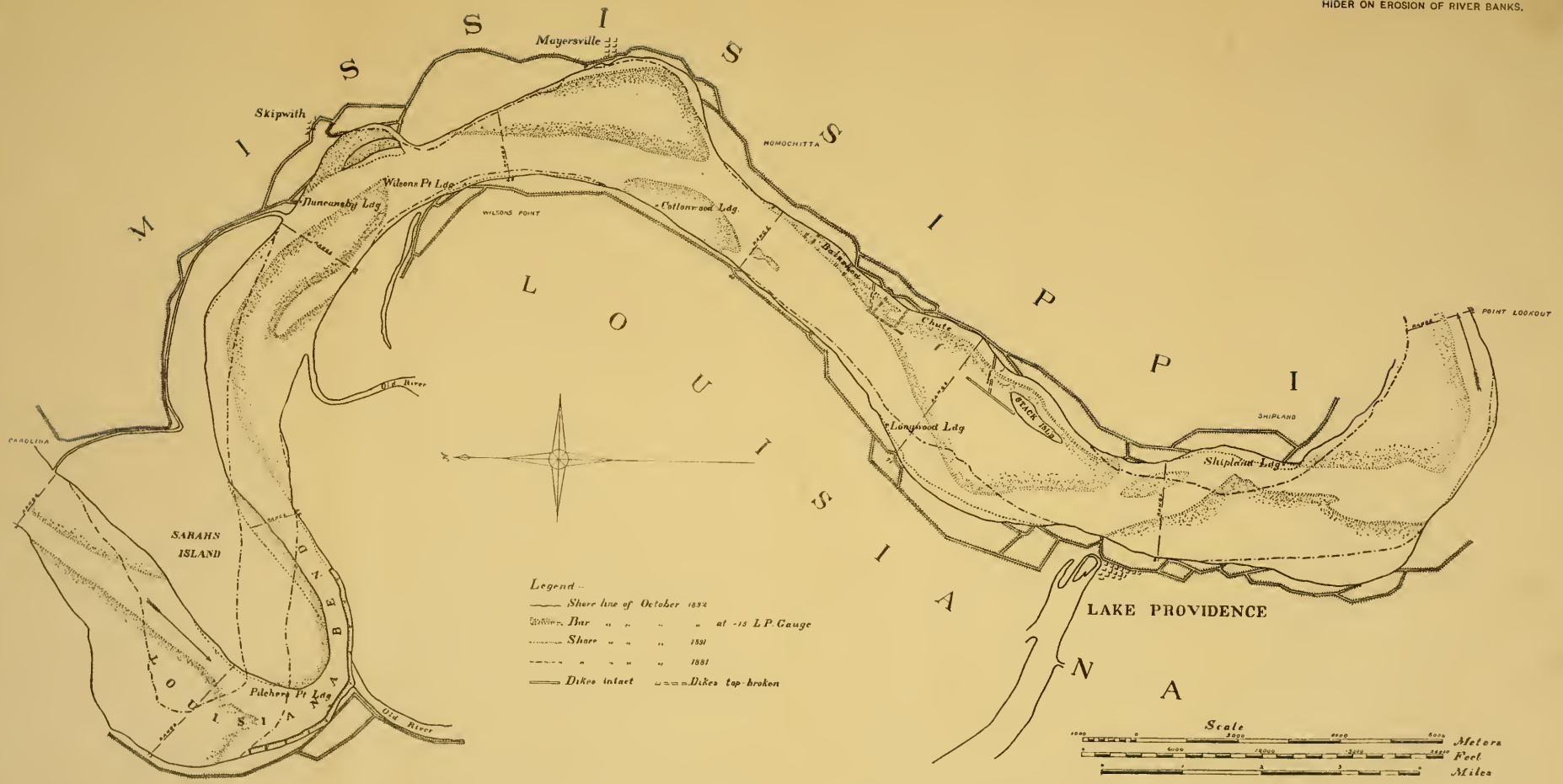
HIDER ON EROSION OF RIVER BANKS.



still the lateral movement as a whole is much less than has generally been supposed; in fact, the variation in the position of the bed of the river from year to year is confined within comparatively narrow limits. It is to be regretted that the absence of fixed reference points in the earlier surveys prevents a closer comparison. The results given, based on the most accurate data obtainable, have been collated, not without an immense amount of care and labor, and great credit is due to the writer of the original paper for presenting such a mass of valuable information in so clear and comprehensive a manner.

While the preponderance of evidence leads to the conclusion that the river has remained for several generations, and will probably continue, in about the same relative position in the valley, still, the lateral movement is so great from year to year that the problem for the final improvement of the river resolves itself into giving permanency and stability to the banks and confining the active forces of the river to the bed of the stream itself, thus preventing its energy being expended in undermining its own banks. When this is accomplished it is believed that the shoals at the crossings will disappear and the channel at these places, even at the lowest stages, will have both sufficient width and depth to meet all demands of commerce.

While it is true that caving is generally most active on a falling river, at about mid-stage and below, it is nevertheless a fact that in some localities, at times, the caving is most rapid during high-water stages. At Greenville, Miss., the bank recession during the high-water stage of 1890 for two miles or more above the town, due to erosion and caving, was an average of 500 ft., being 1 100 ft. at one point; this caving took place at flood stage in less than three months.

High-water caving is not infrequent on the Mississippi River between White River and Vicksburg, and is often so rapid as to destroy the levees, occurring as it generally does where it is not anticipated. This high-water caving is probably due principally to erosion caused by changes in the direction of the current above, the main energy of the stream being directed to a particular point of attack, while the current is at its maximum velocity.

A map (Plate I) of what is known as "Lake Providence Reach," beginning at Carolina Ldg., 517 miles below Cairo, is submitted as giving additional data. Careful surveys under the direction of the Mississippi River Commission have been made from year to year on this

portion of the river, based on the triangulations referred to in the original paper. A comparison of the surveys indicates a gradual increase of high-water widths due to caving.

These surveys were plotted to a scale of 1 to 10 000, and the widths measured by scale.

Length of river compared, 7 miles.
Mean high-water width, 23 sections compared.

Ranges 39 to 61. Date of survey.	Feet.
1882	5 824
1883	5 802
1884	5 882
1886	5 909
1888	5 962
1890	6 109
1891	6 184
1892	6 452

Length of river compared, 6 miles.
Mean high-water width, 21 sections compared.

Ranges 61 to 81. Date of survey.	Feet.
1882	6 005
1883	6 048
1884	6 057
1886	6 299
1888	6 595
1890	6 833
1891	6 962
1892	7 370

Comparison of Surveys L. P. Reach From 1883 to 1892, Excluding Chutes.

68 ranges compared. Distance, 22½ miles.	Year.	Mean low-water width.	Mean areas.	Mean maximum depths.	Mean depths.
Range 14 to 81	1883	2 222	44 859	41.5	20.2
" 14 to 81	1884	2 328	50 141	44.9	21.5
" 14 to 81	1885	2 459	50 506	42.5	20.6
" 14 to 81	1886	2 595	50 888	40.3	19.7
" 14 to 81	1887	2 731	51 100	38.1	19.7
77 ranges compared. Distance, 25½ miles.					
Range 5 to 81	1888	2 900	58 906	40.2	20.3
" 5 to 81	1889	2 787	54 759	38.8	19.6
" 5 to 81	1890	2 673	50 036	37.6	18.8
" 5 to 81	1891	2 197	41 873	34.3	19.1
" 5 to 81	1892	2 255	43 108	38.4	19.1

INCLUDING CHUTES.

68 ranges compared. Distance, 22½ miles.	Year.	Mean low-water widths.	Mean areas.	Mean maximum depths.	Mean depths.	Mean high-water widths.
Range 14 to 81	1883	2 423	51 869	43.2	21.4	5 471
" 14 to 81	1884	2 588	52 310	45.4	20.2	5 518
" 14 to 81	1885	2 701	40 627	42.9	15.1	5 642
" 14 to 81	1886	2 812	43 008	40.4	15.3	5 771
" 14 to 81	1887	2 911	40 653	38.0	13.9	5 906
77 ranges compared. Distance, 25½ miles.						
Range 5 to 81	1888	2 877	49 896	40.1	17.3	6 001
" 5 to 81	1889	3 086	56 645	38.8	18.3	6 146
" 5 to 81	1890	2 929	52 535	37.4	17.9	6 287
" 5 to 81	1891	3 056	44 292	38.0	14.5	6 588
" 5 to 81	1892	2 766	45 347	38.3	16.4	6 737

The maximum bank recession due to the caving at any one locality in the 10 years was in the vicinity of Pilcher's Point, and was 4 500 ft. It will be noticed at Sarah's Island that the river has moved to the west more than the width of itself in the 10 years covered by these surveys. By reference to the maps submitted, it will be observed that at nearly all localities where caving has been most active in a bend there is a large sand bar on the opposite side of the river.

Caving of the banks in concave bends generally takes place from the following causes:

The yearly accretions made to the bar on the opposite side during each successive high water, together with the deposits made during the high water in the bend itself, have the effect of decreasing the area and capacity of the previous low-water channel. As the water begins to fall and mid-stage is reached, the channel capacity at the particular point is not sufficient to pass the flow.

To re-establish a water-way of the proper size, scour takes place along the concave bank as offering the least resistance to erosion, a widening of the channel and a consequent undermining and caving of the banks follows, as the water continues to recede to a lower stage, until equilibrium is restored, when the caving for the time ceases.

The continued recession of the bank lines in bends, due to caving, and the consequent changes in the channel below, preclude the adoption of any system of permanent improvement not having for its main feature stability and permanence of the bank itself; any other plan will fail to give satisfactory results.

It may be of interest to state briefly what has been done recently in this direction in revetting parts of four of the worst caving bends between the mouth of White River and Vicksburg, and the measure of success achieved. This work has been done under the supervision of the Mississippi River Commission, an officer of the Corps of Engineers having direct charge of the improvements. This portion of the river is selected on account of the writer's familiarity with the work, having been in local charge of the improvements when they were made.

Before giving a description of the work, it may be well to state briefly the plan of construction adopted, which was essentially the same for all the different works. The method of procedure was as follows:

First.—To grade to the water's edge, at low stage, the nearly vertical banks, which are from 30 to 40 ft. above low water, to a slope of 1 to 4 by the use of a hydraulic grader specially designed for this purpose, filling up the holes and washouts with scrapers and by hand until the slope was fairly uniform.

Second.—To construct mattresses of willow brush and poles from 2 to 4 ins. in diameter, and from 30 to 50 ft. long. These mattresses were built on boats constructed for the purpose. The mats, while being built and sunk in place, were held out in the river from mooring barges, placed at right angles to the bank, the mooring barges and upper end of the mat being held in position by wire cables securely fastened to proper anchorages on shore.

Third.—After the mat was constructed, usually from 500 to 1 200 ft. in length, and from 250 to 300 ft. in width, it was sunk in place by first distributing over the entire surface sufficient stone to destroy its buoyancy; then barges of stone were floated across the head of the mat, just below the mooring barges, and enough stone thrown on the mat to sink the upper 100 ft. to the bottom; the barges were then allowed to drift slowly down stream, stone being thrown on the mat until it was in place on the bottom. The usual quantity of stone used on these mats was ½ cu. yd. of loose stone to each 100 sq. ft. of surface.

Fourth.—After the mat was in position on the bottom of the river, the upper-bank slope from the low-water to the two-thirds stage was paved with rip-rap, laid closely by hand, to an average depth of about 10 ins.

The average quantity of material used for each 100 sq. ft. of mat-

tress was .87 cords of brush and poles, and 0.5 cu. yd. rip-rap. For paving on the upper slope, 4.5 cu. yds. of rip-rap per linear foot of bank.

The average cost per linear foot was $28, to which should be added, for repairs to plank and deterioration of same, and surveys, etc., 25%, making the cost per linear foot of completed work about $35.

Lake Bolivar Front Revetment, 417 miles below Cairo.—The object of this revetment was to prevent the further caving of the bank in front of the Lake Bolivar Levee, one of the largest levees in the State of Mississippi, built several years ago, and crossing the head of Lake Bolivar. The main current of the river set against this bank, and the caving had been rapid for several years previous, which, if allowed to continue, would in a short time have destroyed this large and important work, requiring a new location of the levee system behind the lake and leaving a number of valuable plantations without levee protection.

This revetment was built in the fall of 1888. The entire length of the bank revetted was 4 250 ft.; on the upper 3 300 ft., the mats were 250 ft. wide; the last mat, 1 100 ft. long, was only 180 ft. wide. The brush revetment was made continuous to the top of the bank, and afterwards covered with stone sufficient to keep it in place, to which has been added on the upper bank-slope, in the way of repairs, additional stone to make the slope about 10 ins. in thickness.

The reduction in width of the lower mat from 250 to 180 ft. was necessary on account of the high stage of water, rising river, and consequent rapid current and drift accumulations at the time it was constructed. The expectation was to reinforce this narrow mat by sinking one of the full width on top of it the next season. No further work, however, has been done.*

The high stage of water at which this last mat was sunk prevented it extending as far out in the river as it would have done had the work been built during the usual low-water season. The outer edge of the other mats extended from 120 to 130 ft. farther out.

From a careful survey recently made, the mats, where of the full width of 250 ft., were found intact; the narrow mat was found displaced and broken. The set of the current is still against this bank, but no

*For a more detailed description, reference is made to report of Mississippi River Commission contained in report of Chief of Engineers, U. S. A., 1890–92.

damage has been done to the upper end of the work, where the mats are of full width, nor is there any scour shown either through the body of the mats or beyond their outer edge. The work can be considered quite satisfactory.

Ashbrook Neck Revetment, 446 miles below Cairo.—This work was designed to preventa threatened cut-off between Georgetown and Miller's Bend; the distance across the narrow part of the neck is only 1 800 ft., while the distance following the river is about eight miles. This revetment was begun in the fall of 1890, the caving at the time being quite rapid, and was continued in 1891 and 1892 until 9 400 ft. of bank was revetted; mats, from 250 to 350 ft. wide. At the close of the season's work in 1890, a narrow mat 180 ft. wide and 500 ft. long was built for similar reasons for building the one at Bolivar; this mat did not extend out far enough to protect the under-water slope from cutting during the high water immediately following, and the ensuing season a mat of the full width of 300 ft. was laid over this.

Careful soundings taken over this work in November, 1892, show the work done in 1890 and 1891 in good condition; there are no signs of undermining at the outer edge of the mats, nor has there been any slipping or breaking down of the upper bank-slope. This revetment may be considered a success, as the caving of the upper side of the neck, where the revetment was built, has entirely ceased.

Greenville Harbor Revetment, 477 miles below Cairo.—During the high water of 1890, the river began caving at an alarming rate; at one point the recession of the bank was 1 100 ft. during three months, caving into the river three separate lines of protection levees, and jeopardizing the existence of the upper part of the town of Greenville. The bank here is almost entirely of sand, except for a short distance below the natural surface, and erodes rapidly.

The design of the revetment was to prevent a further deepening of the bend by stopping the caving, and thus offer protection to the levee system and the town itself. Six thousand six hundred feet of revetment was built in the fall of 1891; this was extended up stream, in the fall of 1892, 4 450 ft., making 11 050 ft. of bank protected by revetment; mats, 300 ft. wide.

A careful comparison of surveys made immediately after the construction of the work, and that of April, 1893, shows that the entire work is in good condition, although a general scour or deepening of

the channel is shown along the outer edge of the mats. From 1891 to 1893, two high waters intervening, the average scour was 11 ft.; from 1892 to 1893, one high water intervening, 3 ft. In other words, the average scour the first year was 8 ft., and the second year, 3 ft., or less than one-half.

Louisiana Bend Revetment, 523 miles below Cairo.—The purpose of this improvement was to prevent further deepening of the bend, which had moved westward, in the last 10 years, 4 500 ft. more than the entire width of the river itself. The continued caving caused changes in the channel of the river below, which it was desirable to prevent.

In 1889, 6 034 ft. of revetment was constructed; in 1891, the revetment was continued down 5 224 ft., and in the fall of 1892, 5 835 ft. additional was built, making the entire length of the revetment in this bend 17 583 ft. The work done in 1889 is all in place, being now protected from the direct action of the current, except at high-water stage, by a sand bar formed since the work was built; the work of 1891, so far as can be determined from comparison of later with former surveys, is in good condition. Surveys made over the revetment of 1892, both before and after the mats were in position, and during the three months following, without, however, an intervening high water, indicate that on many of the sections a scour was taking place, not only along the outside of the mat, but through the body of the mat itself.

The Bolivar and Ashbrook Neck revetments show no scour, either through the mats themselves or at the outer edges. At Greenville and Louisiana Bend, scour, both through the mats and immediately beyond their outer edge, has taken place. Why this should be the case in the two latter instances, and not in the former, can only be accounted for by the existence of different conditions, as the mats at both Greenville and Lousiana Bend were of full width of 300 ft. As these four revetments were constructed in the most rapidly caving bends, where the lateral movement of the river was greatest, it would be expected that the resistance offered by the revetment in preventing erosion next to the bank would naturally result in a scour or deepening of the channel beyond the outer edge of the mats.

It would seem that the maximum rate of scour occurs after each high-water stage, and if the revetment has sufficient strength to withstand the attack until the necessary enlargement of section takes

place, there will be a gradual shifting of the line of deep water toward the bar on the opposite side and an eroding of the bar accretions deposited at preceding high waters.

As the natural result due to holding the bank in a caving bend is a steepening of the slope between the bank line and deep water, the importance of having a revetment so constructed as to permit a limited scour through it while these changes are in progress, and at the same time to have pliability enough to adjust itself to the changed conditions, is evident; at the same time the revetment should be constructed of such material as is lasting, so that no renewals will be required.

The work cited above shows what has already been done in the direction of holding some of the worst caving banks on the Mississippi River. It is too early to state positively as to the permanency of the works; so far they promise satisfactory results.

Improvements in methods will continually be made, as experience is gained as to the proper treatment of this river, and future engineers, profiting by our mistakes, will eventually solve the problem, how to prevent the "Erosion of the River Banks on the Mississippi and Missouri Rivers."

Samuel H. Yonge, M. Am. Soc. C. E. (by letter).—It has been my practice for many years, while in charge of the construction of works of improvement on the Missouri River, to compute the volume of erosion in caving bends in the vicinity of my work. I have found the average height of the prism of erosion to be about 35 ft. for bends, and about 27 ft. for towheads and first benches. As the greater part of the erosion occurs in the bends, possibly as much as 75%, the figures given by Mr. Ockerson for erosion of Missouri River banks are probably close approximations. The character of the 140 miles of the Missouri River between Sioux City, Ia., and the mouth of the Niobrara River being the same as that of the river below Sioux City, it appears proper, in estimating the volume of erosion, to apply the annual rate determined by Mr. Ockerson to 942 miles of river. This would make the total quantity of material washed into the river from caving banks about 702 670 000 cu. yds.

Above the mouth of the Niobrara the Missouri Valley is narrower, and the bank erosion, for this and other reasons, is less than below that point. This deficiency, however, is probably made up for in a great measure by the eroded material and surface washings brought in by the

upper tributaries. The extent of these contributions, as well as of those coming from tributaries below Sioux City, is not known. It is, therefore, obvious that the total quantity of material washed into the Missouri is far greater than estimated, and this view is sustained by the following:

It is found, by sounding the same cross-section of the river at all stages, that the section is deepened by scour during floods, and filled back during the lower stages, when bank erosion is greatest.

The extent of the scour and "fill-back" varies in different localities and for different conditions of flow.

From data collected at several points between Sioux City and the mouth of the Missouri, it appears that the change in area of cross-section on account of "fill-back" is from 3 000 to 8 000 sq. ft.; 5 000 sq. ft. appears to be about an average. This would amount to about 978 000 cu. yds. per mile, or over 30% greater than the estimated bank erosion. The excess of "fill-back" over bank erosion is probably supplied by tributaries. The "fill-back" is set in motion whenever the stage is higher and the current swifter than that at which it was deposited. The distance it is carried each year, and its rate of travel at any given stage, cannot be directly determined.

As the average annual volume of bank erosion is apparently constant, and as new river banks do not build up any more rapidly than the old ones are eroded, and, furthermore, as the general elevation of the thalweg is practically the same from year to year, it is to be inferred that the volume of eroded material that is swept into the Mississippi each year from the Missouri and its tributaries is not less than that of the "fill-back."

A series of sediment observations made on the Missouri at St. Charles, Mo., about 20 miles above the mouth of the river, in 1879, shows the total volume of sediment carried in suspension to be about 188 900 000 cu. yds., or about 20.5% of the whole volume of "fill-back," and about 27% of the total bank erosion between the Niobrara and the mouth of the Missouri.

During a part of the period that the above sediment observations were being made at St. Charles, viz., from the middle of February till the latter part of June, similar observations were made for the Mississippi at Columbus, Ky.

By allowing a time interval of three days between St. Charles and

Columbus, it was found that about the same volume of sediment observed passing St. Charles also passed Columbus.

If the total volume of eroded material that is brought into the Mississippi is to be considered, and a comparison instituted between it and the quantity of sediment passing New Orleans, the material contributed by the Missouri and the reach of the Mississippi between the mouth of the Missouri and Cairo should also be taken into account. Assuming for the latter the same annual rate as for the Missouri would make the total volume of eroded material that annually goes to build up the delta of the Mississippi equal to about 1 727 500 000 cu. yds. This quantity is still further increased by material from the Upper Mississippi, the Missouri above the Niobrara, the Ohio and the lower tributaries, for which data do not exist or are not at present available. Accepting this view, the sediment observed passing New Orleans probably represents not more than about 16% of the whole volume of erosion.

The failure of sediment observations to account for anything like the whole volume of eroded material may be explained by the fact that only the finer particles of silt are carried in suspension by the water, and thus come within the range of actual observation, the coarser materials being rolled or moved along the bottom.

Humphreys and Abbott estimate the total volume of material "pushed" into the Gulf annually at 750 000 000 cu. yds. They also state that the river water is distinguishable in the Gulf 20 to 25 miles beyond the bar during flood stages, and from 5 to 10 miles at low stages.

In conclusion it is to be understood that the figures I have given are only estimates based on available data. A complete and satisfactory discussion of the subject would require precise data covering the whole river and its tributaries, especially of those tributaries flowing through the "Bad Lands," from which a very large proportion of the silt is derived.

Henry B. Richardson, M. Am. Soc. C. E. (by letter).—Mr. Ockerson's paper presents a large body of facts in an interesting field for investigation which appears heretofore not to have been very thoroughly explored, and it suggests inquiries which may hereafter lead to important conclusions as to the proper methods of improving the Mississippi, Missouri and other alluvial rivers.

The establishment by the Mississippi River Commission of the "stone lines" described in the paper affords a base for the continuance of investigations as to the amount and rate of bank erosion in the future, and it would seem that a great deal of fairly accurate data as to bank erosion in the past might be obtained by comparison of the land surveys, which were generally made as much as 50 years ago, with the more recent surveys. It would materially add to the interest and value of the Mississippi River Commission's maps if the section lines of the land surveys were shown on them. The river banks, as they existed at the dates of the land surveys—especially on the abrupt, concave sides of the river—could certainly then be retraced with close approximation to truth upon the later maps. The inaccuracies of the original surveys should not greatly affect the result, as a reconstruction of the parts of subdivisions lost by erosion would start from the accurately determined positions of the "back corners" that remain.

It has been demonstrated that all streams in north latitude should tend to crowd upon and abrade their right banks more than their left, and it is frequently asserted that they do so. The explanations of rapid erosion sometimes given when it happens to be on the right bank of the river, as being due to this cause, must always have appeared more fanciful than real to anyone who looked much beyond the immediate locality of such erosion; but Mr. Ockerson's tables show us that whatever theoretical value may be allowed to this tendency of rivers—and railroad trains as well—to "turn to the right as the law directs," it has only negative results on the Mississippi and Missouri rivers when taken together, as the aggregate area of erosion on their right banks is shown to be less than 87% of that on their left banks.

While it is undoubtedly true that the amount, and especially the rate, of lateral movement of the banks of these rivers is popularly much exaggerated, it is evident that, if continued indefinitely, at even the moderate average rate, as deduced from Mr. Ockerson's tables, of only about 90 ft. per annum, the bends would finally develop to the right and left until they might occupy the entire width of the alluvial valleys, were it not that long before such limits could be reached they must inevitably inosculate and form "cut-offs." There are indications far inland from the present banks of the Mississippi that its channel, or the channel of some large arm or branch from it, had its bed at one time or another in almost every part of the alluvial valley. Many

of the curved lakes and sloughs that abound in the interior present all the characteristics of the old river cut-off lakes, whose origin is undoubted, and only differ from the more recently formed lakes in being more generally silted and overgrown with cypress trees, and in remoteness from the present course of the river.

A somewhat curious case of repeated cut-offs in the same place appears to have occurred at Lake Concordia, just above Natchez. It would appear from an inspection of the map that two successive cut-offs have already taken place here, and that a third is now threatened; and that the location of both of those that have already occurred must have been in almost exactly the same latitude as that now threatened, and not more than a mile or two distant.

If cut-offs could be entirely prevented, it seems a logical conclusion that the river must in time attain such a length and consequent diminution of slope that all erosion due to the direct force of the current would finally cease. But the operation of the natural forces that produce bank erosion has undoubtedly been producing such cut-offs from time to time during long-past ages, though there are not wanting instances, besides those near the mouth of Red River mentioned in the paper under discussion, in which the hand of man has assisted Nature, or at least hastened the accomplishment of her work. Among these may be noted the Palmyra cut-off of 1867, where a ditch had existed for years previous, which had been gradually enlarged, so that the year before small steamboats had passed up through it in time of high water. So, also, the cut-off of 1884, at Cole's Point—although the paper seems to intimate that the place where the river broke through was not on the line of the ditch which had been cut there years before—was certainly of a width, when first observed as a cut-off, that extended about equally on either side of the line where the ditch had been. This ditch, by the way, had been slowly enlarging for a number of years, though a cut-off was not immediately apprehended previous to the time it took place. In 1878 a measurement of this ditch showed it to be 60 ft. wide and 15 ft. deep about 100 yds. from its lower end, though considerably narrower near its upper end. The well-known canal cut by General Grant in 1863 across the point opposite Vicksburg still remains well inland from the bank of the river, as cut off in due course of natural causes in 1876, and probably was never really expected to accomplish the purpose for which it was ostensibly made.

Mr. Ockerson shows that the maximum rate of erosion on the Mississippi lies in the stretch of about 150 miles between Grand Lake, Ark., and Waterproof, La., and states that "the area of greatest caving coincides with the leveed portion of the river." But, as only about 65 miles of this stretch of maximum erosion has levees on both sides, while the remaining 85 miles does not appear to differ in its proximity to the hills from the stretch of about 175 miles next below, where the rate of caving becomes much less, it hardly seems a coincidence that should carry any such weight of suggestion regarding an "intimate relation" between the levees and the caving banks as would have arisen had the leveed part of the river and the area of greatest caving been coterminous as well as coincident. Greater velocity of current and greater range of stage should be found in a leveed stretch of river than in an otherwise similar unleveed stretch, and, therefore, greater erosion of bed and banks might be expected; but there seems to be no direct proof of it in the coincidence referred to.

The author of the paper also notes that timber growing on the banks has no appreciable effect in arresting caving, and that excessive caving and excessive curvature are rarely coincident. What, then, are the causes of the excessive bank erosion along this 150 miles of river?

It is shown that on the Missouri River the annual area of erosion per mile of river is greater than on the Mississippi by over 40%, and that "in general the banks of the Missouri are composed of fine sand and silt," which may indicate that this character of material in the banks is a cause for their greater erosion—a conclusion which might suggest itself as not unreasonable on other grounds. If we take Table No. 1, and assemble the several items according to their classification under the column headed "Character of Banks," putting in one class all such items as are classed "sand," "sandy," "mostly sand," "sand and silt," etc., we find that the average rate of caving for such banks amounts to about 30.3 sq. yds. per linear yard of caving bank, while the average for all classes of caving bank is only 24.2 sq. yds. We also find that the length of caving bank of this sandy character is greater per mile of river in the 150 miles subject to excessive erosion than in other parts of the river, being at the rate of 718 yds. per mile; while in the stretch of 510 miles above it is about 503 yds. per mile, and in the 225 miles below only 462 yds. per mile of

river. The result of this comparison, as far as it goes, tends to the conclusion that one of the causes of excessive erosion in the 150 miles of river between Grand Lake and Waterproof may be the excess of sandy banks in this stretch. But, while the average annual rate of caving of sandy banks, as above stated, is 30.3 sq. yds. per yard of caving bank in the whole 885 miles covered by the table, the average annual rate in the 150 miles subject to excessive caving is 55.4 sq. yds. per yard of caving bank, which would indicate that the excess of sandy bank is not the only cause of excessive erosion in this stretch.

Mr. Ockerson notes as a probable cause of excessive erosion the increase of slope due to the Cole's Point cut-off in 1884. But if we go back about 19 years and consider the relative length of river channel then, and its length in 1891, in the stretch of 150 miles of excessive caving, there appears still greater probability that increase of slope may play an important part among the causes of excessive caving in this stretch. Besides the Cole's Point cut-off—the lower end of which is only about seven miles below the limit of the 150-mile stretch considered—there have occurred within the limits of the same stretch, since 1865, the Terrapin Neck cut-off in 1866, the Palmyra cut-off in 1867, and the cut-off opposite Vicksburg in 1876, besides a number of cases where the main channel of the river has forsaken the longer course it followed in 1865 and taken shorter chutes, as at Islands 95, 97, 103, Diamond Island Towhead and Grand Gulf Island.

It is not difficult for any one familiar with the river during the past 30 years to trace a line on the maps published by the Mississippi River Commission that will represent with approximate accuracy the center of the main channel as it was in 1865. The distance along such a line—giving it the greatest length possible where there is a reasonable doubt as to its exact position—from Grand Lake to the lower end of the Cole's Point cut-off is 205 miles. The length of channel line between the same points in 1884 was 157 miles, showing a decrease in length of over 23%, with a proportionate increase of slope.

The development of channel line in the stretch of river from Cairo to Grand Lake is about 1.76 times the length of the shortest line between the two points, and in the stretch from the lower end of Cole's Point cut-off to Donaldsonville, it is nearly the same, or, say, 1.82 times the length of the most direct line down the alluvial valley. But in the intermediate stretch of excessive erosion between Grand Lake

and the lower end of Cole's Point cut-off, the length of the river is now only about 1.67 times the length of the most direct line down the valley between the termini, while in 1865 it was at least 2.18 times its length.

One of the probable causes of caving mentioned in the paper under discussion is the return flow (when the rivers fall) of the water that has penetrated and saturated the banks while the river was at a high stage. This effect would evidently be somewhat proportionate, other things being equal, to the range between high and low water, and therefore, in a general way, should be smaller below the mouth of Red River than above, as the oscillation becomes less from that point down to the Gulf than it is above. But in the 150-mile stretch of excessive erosion, the oscillation between high and low water stages is not materially different from that of the other stretches included in Mr. Ockerson's table, and if the greater oscillation of the upper part of the river has any influence in producing greater erosion in this 150-mile stretch than in others immediately above and below it, it can only be in consequence of the more sandy and porous character of the material composing the banks.

If we ascribe the excessive erosion in this part of the river solely to the character of the material composing its banks, there seems no reason to assume that the locality of excessive erosion will move either up stream or down. If, however, we consider it a result of recent shortening of the channel, and consequent increase of slope, we might conclude that at some future time, when a normal length of line, and a slope adjusted to the character of the bank material has been recovered, the region of maximum erosion may move elsewhere; but that it would more likely move up stream than down, because the apparent opportunities for cut-offs are generally greater and more numerous above than below.

As stated in the paper, "It is plainly evident that bank erosion is one of the principal factors in the solution of the problem of river improvement and control." If the banks in the concave bends of the river can be revetted and held in any fixed position, the opposite banks will tend to adjust themselves to concentric lines, leaving a channel of the requisite width to carry whatever volume of water the river may have between banks. The energy of the current need not, then, be continually expended in removing and rearranging the material brought down by the caving banks, but may attack and disperse

the shoals and bars which now obstruct navigation. This material, once pushed aside and deposited without the channel, would largely remain where it was placed, and not be—as it is in a state of Nature—forever wrought over and over.

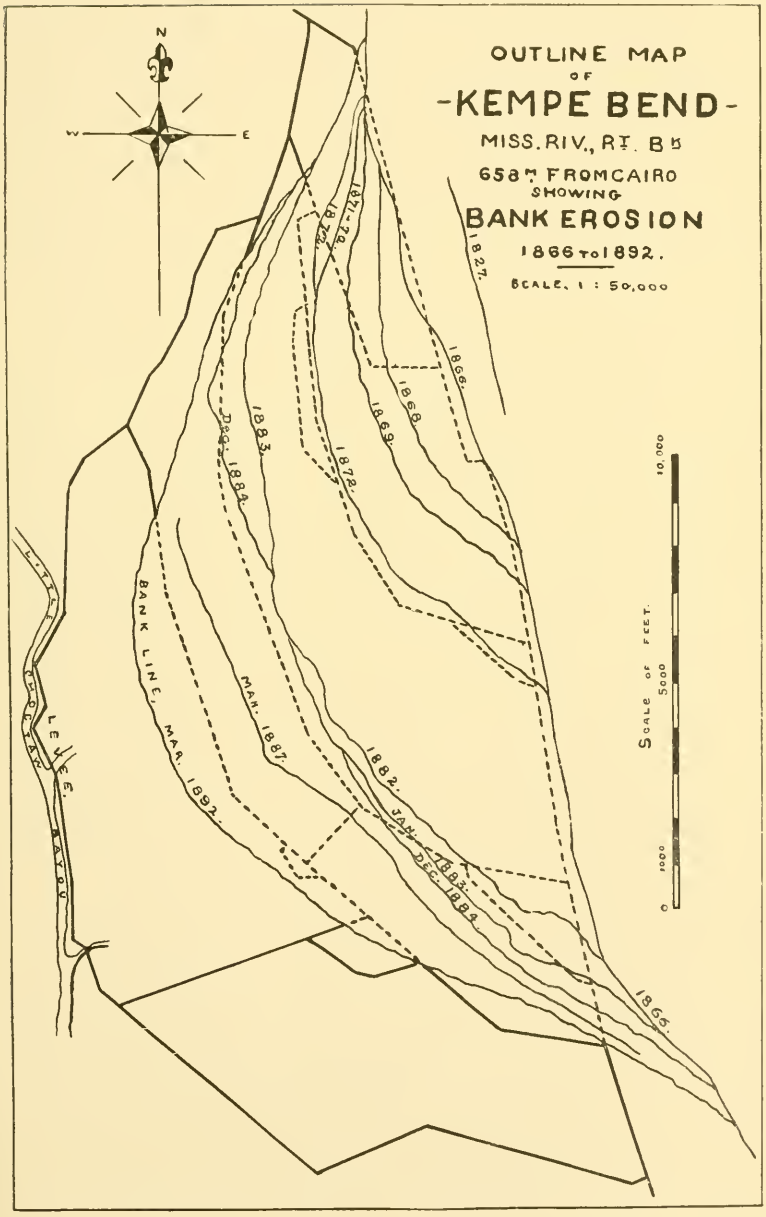
If the banks were fixed in position, there would no longer remain a doubt as to the ease and certainty with which floods could be restrained from overflowing the adjacent lands, as embankments of any size and strength desired could then be built and continuously improved, instead of being, as now, subject to the perpetual encroachment of the caving bank, and the consequent necessity for frequent renewal upon new lines.

As an example of the enormous expenditure of means and labor that is sometimes involved in levee construction consequent upon bank erosion, the outline map of "Kempe Bend," on the Mississippi River, is here presented (Plate II). As will be seen, this stretch of about six miles of the right bank of the river was not a bend at all in 1866, but rather a reach slightly convex to the river. The position of the bank at various dates since 1866, taken from records in the office of the Louisiana State Engineers, is shown on the map, and that of the numerous lines of levee built since that date. The area caved into the river from 1866 to 1892 is about 2 900 acres, involving a great part of three large and productive cotton plantations. The construction of levees made necessary from time to time by the recession of the bank has thus far involved over 2 500 000 cu. yds. of earthwork, at a cost of nearly $1 050 000.

The most difficult and expensive bank revetments on the Mississippi River, have, perhaps, cost as much as $150 000 per mile. But if the two miles, or less, of bank in this bend, which in 1866 comprised the limits of erosion, could have been revetted and held from that time to this, at this cost per mile, there would have been a clear saving of nearly $700 000 in levee construction alone, not to speak of the value of the plantation property lost since then. And even the entire length of about six miles now undergoing active erosion might be revetted at the same rate with a probable margin of profit hereafter.

Horace M. Marshall, M. Am. Soc. C. E. (by letter).—While testifying the highest appreciation of the value of the information contributed concerning "Erosion of River Banks on the Mississippi and Missouri Rivers," yet with greater emphasis must it be said that, to be

PLATE II.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 687.
RICHARDSON ON EROSION OF RIVER BANKS.



presented with so many figures and facts at the finger ends of a prominent engineer who has for a number of years made a special study of river regimen is rather disheartening when not a single dogma is enunciated. The "locality of excessive erosion" seems affected as was the sleepy-headed Irish nurse's patient, "wan toime he is the wan way and thin he is anither way, and thin he is the wan way."

Column 3 in Table No. 1 possibly would have appeared more germane to the subject and more closely related to the column of character if the numbers had placed under them for denominators the distances from the top of the bank out to where the depths obtained, for it is usual to estimate the stability of earth according to its angle of repose rather than its height.

The table would be added to in more than one way by a new column of water-surface slopes, or, better still, by adding also maximum velocity in each reach. Whatever the cause of caving banks, it is quite certain the caving would stop if the caved-in material was not swept away by the onflowing water. It is not meant that maximum slope will be found concurrent everywhere with maximum bank erosion, even though the bank material be the same, for, since change of direction involves loss of force, a greater curvature may counteract the greater slope, or a less mean hydraulic radius have the same effect. Since the variation in bank material of an alluvial river is more local than regional, and the last column of Table No. 1 is cited to bear out the assertion, it follows that the reason of caving which occurs in particular districts must be looked for aside from bank constituents. While the power of the water to do work depends on its velocity which, other conditions being equal, varies with the slope, yet, if the slope be excessive, the scour will not be as great nor the channel as deep as obtains in places of less slope; because the velocity, being due chiefly to slope, is more nearly uniform at all depths, both close to shore and in the channel; hence the scour at the foot of the bank causes caving and change of form of cross-section, which lessens the velocity, or if the current strikes the bank at an angle, the river is moved laterally before sufficient time has elapsed for deepening the bed. In the other case, the velocity, being due principally to depth, is less near the bank, and before the caving begins the channel is scoured and the slope flattened.

Inasmuch as an alluvial river makes its own bed, the slope and

not bank constituents determines its cross-section. So long as its discharge remains constant, the banks must remain stable, but any increase in the flow must bring about caving which will lengthen the river or change the cross-section. Thus it is readily seen how levees cause caving banks. That the caving is not excessive at time of high flood is largely due to the current line withdrawing from the bank towards midstream, as the jetty action of the bars on the convex bank becomes proportionately less with the increase in volume of the river. Scour is going on mean time in the bed, and with the fall of the water this increased depth is carried, by the again bending current, in close to shore, and the banks are undermined. It is not intended to depreciate the effect of the seepage from the banks, but it is hardly to be credited that seepage occurs only in rare instances from opposite banks at the same time. The drainage from the land is everywhere as the water falls, but caving only occurs where the current is close along shore, and usually where there is vortex motion due to the water impinging on the bank. On the other hand, caving does occur when there is no drainage from the land, for beside the many cases of caving at high stages which escape notice by reason of bank submergence, but which are attested by the floating foam on the water, instances are not few where levees have caved into the river at extreme flood, and probably every cut-off takes place near that time; certainly when the water is over its banks.

Turn now to Plate XLII, *Transactions*, Vol. XX, page 228, and imagine a third line nearly equi-distant below and drawn nearly parallel to the low water-line there shown, but somewhat divergent toward the lower right-hand corner; then consider where the depth at high water (indicated by this line) multiplied by the slope would give the greatest product, velocity.

So much for the whence of bank erosion, and now for the whither. Inasmuch as there were 916 miles of caving banks on the Mississippi in 885 miles of river, and 818 miles of caving banks on the Missouri in 802 miles of river, it is certain that the former widened along 41 miles of its length, and the latter along 16 miles of its length. If "it may be accepted as practically true that erosion on one bank is followed by an equivalent accretion on the other," or, "in some cases the resultant of the two is a very large decrease in width," yet the burden of evidence seems to show that widening of the river contributed some

"of the solid matter passing New Orleans, computed to be about 10 square miles, 26.8 ft. deep." If this is the measure of the widening, and all the rest of the material from caving banks is taken up by accretion on the opposite shore, some other source of supply must be drawn on for the sediment passing New Orleans. The widening on the Mississippi along 41 miles of average bank would cover 10 square miles 3.7 ft. deep, and the Missouri contribute but 1.1 ft. more; and since the Upper Mississippi, Ohio, Arkansas and Red rivers cannot individually do more, it follows that the river-bed must contribute an amount equivalent to 0.2 ft. deep half a mile wide over the whole of its length of 885 miles.

A glance at Plate L, *Transactions*, Vol. XXVIII, page 398, reveals a notable decrease in the number of islands, notwithstanding the similarity in the location of some. By count the early sketch shows 30 below Les Côtes de Yazous, the later map but 9 below Vicksburg, excluding cut-off islands which do not occupy a part of the river's bed. The numbers in the next stretch to the Arkansas River are 57 and 20 respectively, to Memphis, 49 and 21, and to the mouth of the Ohio, 70 and 23. A considerable number of islands, then, have been carried down to the sea or become merged in the banks by this mighty river, and little more than a century serves to record a clearing of its channel and a concentration of its waters.

The answer to the next and possibly most important question of the whither of the bank erosion involves another: What of the bottom?

The generally accepted belief is that clearing, cultivation and ditching have accelerated the drainage of the water into the rivers; this, with the confining by levees, would produce a greater flow at time of flood which would give rise to increased velocity, which in turn would cave the banks or scour the bed, till the increased frictional resistance or the flattened slope restored the equilibrium. Since the larger river in alluvial soil has the flatter slope, it must be that it is the channel that is deepened by the concentration. The flood height has not been raised to any considerable extent, except locally, so the increase in volume did not take place on top; and yet so short a period as twenty years has prefixed a minus sign to nearly every low-water reading on gauges which were set with zero at "lowest water ever known." It would seem then, that all the solid material brought by

its tributaries, as well as some from its islands, bottom and banks, is cast from its bed or carried out of its mouth.

Cross-sections of Red River, repeated after an interval of 21 years, show, when plotted above the same datum, that the river has widened somewhat and deepened considerably. Level elevations show that the surface at "high water" has risen locally, the line has leveled up, and at "low water" has fallen gradually. The discharge at the same gauge height, during low and medium stages, has nearly doubled. There are no early measurements at time of high water.

To sum up, then: Quickened drainage increases the volume which in turn increases the velocity, and causes caving and scour, which flatten the slope or increase the friction, till the working power of the river and the resistance of its bed are again balanced. The completed river has a larger cross-section, discharges more water with a diminished mean velocity; but because of the increased reservoir capacity of the river-bed, its oscillation may not be altered, its maximum discharge may not be increased, and its period of low water may not be prolonged, even with its greater depth.

Much of this is what is commonly termed theory, but, like most that masks under that title, is only surmise; it suggests, however, a thought of that part of the paper under discussion, where in classic period the author felicitates the future engineer who shall announce the law of water's ebb and flow adduced by present labors. Mayhap, when the time comes round, the future engineer will not adjudge those of this day and generation more prescient than a Humphrey or an Abbot, for while they marked their measure by perishable monuments, we of to-day may be omitting altogether measurements equally important in formulating conclusions.

W. P. Craighill, M. Am. Soc. C. E. (by letter).—Mr. Ockerson has given us a most interesting paper. The few words I shall offer refer to a single paragraph and are simply a caution that care must be taken not to draw too general conclusions from special cases when dealing with rivers, for while general principles and good common-sense in their application must govern in the improvement of them all, a careful and special adaptation of these to each particular case is vitally necessary to prevent unfortunate results.

The constitutions of rivers, if I may use such an expression, while agreeing in general conditions, differ in detail as much as those of

human beings, and when an attempt is made to cure their irregularities, care must be taken to adopt the special treatment suitable to each. The same will seldom answer for any two.

Mr. Ockerson, on page 418 of Vol. XXVIII of the *Transactions*, has the following:

"It is plainly evident that bank erosion is one of the principal factors in the solution of the problem of river improvement and control. If contraction works are built, then erosion becomes active on the opposite bank, and the river soons moves beyond the sphere of influence of such works."

He evidently had in mind the Mississippi and Missouri rivers. They are special and very grand and difficult cases. But his statement is a general one. If, instead of saying, "then erosion becomes active on the opposite side," he had said, "then a tendency to erosion becomes active on the opposite side," I should have agreed with him entirely. And I have very little doubt that he meant what I have substituted for his written words.

In order to show what I intend to imply by the foregoing, I venture to quote from a paper presented to the Society several years ago:

"On the * * * River, the arrangement of the regulating works has been arrived at in the following manner. After the surveys and maps were made and due investigation was had of the requirements of the case, the formulas told what should be the width of channel of an assumed depth and shape, which a river with the characteristics of the * * * would bear. Inasmuch, however, as a river is seldom the same creature for two successive days, and the variations of regimen in a year are large, there are, for these and other reasons, elements which cannot be accurately taken into account by the rigid rules of mathematics. In other words, the best hydraulic formulas give but approximate results. Woe to the man who follows them strictly.

"Under these circumstances, the method of improvement must be to some extent tentative. If one bank of the river is firm and the shape of the reach favorable, spurs may be run out from the other bank, gradually, to nearly the length which would be needed to give the contraction indicated by the formulas. Observation for a year or more may show the need of lengthening all the spurs, to produce the desired effect. Or it may be found they are all too long and must be cut off. Or it may be necessary to lengthen some and shorten some.

"If the bank towards which the spurs run is liable to erosion, it must be revetted, or a training wall be built in front of it. Sometimes it may be best to run spurs from both banks simultaneously. As far as injury from undermining is concerned, the danger for spurs is less

than for training walls, as only the ends of the spurs are thus exposed. When the proper length for the spurs has been determined, their ends may be connected by a longitudinal wall."

A. F. Wrotnowski, M. Am. Soc. C. E. (by letter).—The data incorporated in the valuable paper of Mr. Ockerson bring to mind many important features connected with this great river, whose regimen is, perhaps, considering its importance, least known, not only to the general public, but also to engineers generally, and yet it is a river whose tributaries, stretching 12 000 miles of navigation, permeate and drain the granaries of this country, and which, as a river, is, perhaps, first in importance to the commercial interests of the world. Interesting and valuable as is this paper, it is lacking much in matter to bring out a tangible value for a proper consideration of the subject. A profile of the land surface, of the high and low water-marks of the depths of the river, the slopes, abnormal width, and the wet and dry perimeter of the same, are essential for a proper study to obtain practical results.

It seems to me that the slope of the river bears largely on the development of the forces which cause the abrasions so graphically illustrated by Mr. Ockerson, and that this element should be made an important factor in determining the conditions under which the abrasions are so made. As the slope of the river determines the velocity of the current which is the force or energy causing the erosions and consequent changes, so should this element enter largely into consideration. The slight caving, for instance, as shown in the diagram, which occurs below Red River, as compared with points above, can be accounted for, in part, by the slope of the river from about Red River down to the sea, being less than one-half per mile of the slope above to Cairo.

It is also well known that bank protection and contracting the abnormal widths of the river have been carried on by the United States Government under the able management of the Mississippi River Commission for the last 12 years, and that these improvements must result in considerable influence in bettering the condition of the banks and channels, and that from the remarkable excess of caving in some localities above Red River, as compared with others under about like conditions, shows that some factors are wanting from which to deduct practical results.

J. A. Ockerson, M. Am. Soc. C. E. (by letter).—The discussion of

this paper has brought out many interesting points in a field which is full of intricate problems which challenge the skill of the brightest minds in their solution. The gentlemen who have taken part in this discussion have drawn from their large experience, and have given us the results of their observations in a way that will add much to the value of the pages of our *Transactions*.

In the original paper the author purposely omitted the "enunciation of any dogma," and is quite willing to admit that many "columns" of valuable matter might be added to the tables. The author attempted to give a plain statement of observed facts pertaining to bank erosion. It may be well to remind Mr. Marshall and Mr. Wrotnowski that any one of the items of slope, depth, velocity, etc., would require a very extended paper in their elucidation, and there was no attempt made to cover the whole range of river physics in one paper on "bank erosion."

Mr. Richardson's statement that "a great deal of fairly accurate data as to bank erosion in the past might be obtained by comparisons with the land surveys," is correct. In the surveys made by the Mississippi River Commission it has been the rule throughout, to connect with all reliable township and section corners within the limits of the work that could be readily identified. A great many "corners" of this kind have been located, and, of course, more would be desirable, but definite information concerning them is difficult to get, and any one who has attempted to find them in the tangled brakes of the lower Mississippi will readily understand why they are not more frequent on the maps.

I am quite willing to accept Colonel Craighill's amendment and agree with him that each particular case of regulation requires special study, and often special treatment, to suit the peculiar conditions. These, however, come rather as modifications of the larger general scheme, which involves the regulation and control of the stream as a whole.

Mr. Wrotnowski falls into the common error of supposing that "the slope of the river determines the velocity of the current."

The mean velocity at high water at Hannibal, Mo., 140 miles above St. Louis, is 4.6 ft. per second, with a slope of over 0.7 ft. per mile.

At Columbus: mean velocity, 8.2 ft. per second; and slope, 0.53 ft. per mile.

At Helena : mean velocity, 6.3 ft. per second ; and slope, 0.45 ft. per mile.

At Carrollton : mean velocity, 6.3 ft. per second; and slope about 0.2 ft. per mile.

So the problem is really not as simple as it would seem at first glance.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

688.

(Vol. XXXI.—January, 1894.)

STORAGE AND PONDAGE OF WATER.*

By JOSEPH P. FRIZELL, M. Am. Soc. C. E.

READ OCTOBER 4TH, 1893.

The total annual flow of a stream in Massachusetts is distributed throughout the several months, taking the average of a series of years, in about the following manner :

About 10%	comes	in	the month	of January.
14	"	"	"	February.
20	"	"	"	March.
15	"	"	"	April.
10	"	"	"	May.
4	"	"	"	June.
2	"	"	"	July.
3	"	"	"	August.
3	"	"	"	September.
5	"	"	"	October.
6	"	"	"	November.
8	"	"	"	December.

*Discussions on this paper received before May 1st, 1894, will be published in a subsequent number.

This ratio of distribution is modified to some extent in other parts of New England, the time of extreme high and extreme low water being later in the northern part and earlier in the southern.

This statement exhibits in a very strong light the chief defect of water-power, viz., its extreme variability. An establishment depending wholly on water-power, which would run its machinery throughout the year without serious interruptions, cannot use more than 2% of the annual flow of the stream in any one month, or more than 24% in a year. Even with this use of water it would be exposed to more serious interruptions than appear above. In the months of August and September the flow would naturally be below the average half the time, and often below 2 per cent. In June and October, also, though the flow is 4 or 5% upon an average, there would be many days on which it would be below 3 and below 2. Moreover, the above statement represents the average of a long series of years. In such a series, years will occur with a total flow of not more than half the average, and others with nearly or quite double. Several years will occur in succession, falling 25 to 50% below the average. Others, as much above.

It results from these conditions that no rational use can be made of water-power, subject to the requirements of modern industry, without either steam-power to supply deficiencies in dry seasons, or storage reservoirs to hold back the surplus of the high-water period for use in times of scarcity. With reservoirs it is possible, though hardly ever practicable, to make use of the entire flow of the stream, reducing it to a uniform average stage throughout the entire year, or even for a series, or what may be called a cycle, of years, that is, for a period long enough to embrace all possible fluctuations of the flow. With the aid of steam-power alone it is never possible to make use of the entire product of the stream. This would imply the establishment of wheels and other appliances of water-power to the extent of the maximum flow of the stream, a proceeding obviously irrational. It is not worth while to introduce such plant to a greater extent than can be kept in operation for at least three or four months in an average year. This necessarily leaves large volumes of water to run to waste, so that the use of steam-power does not preclude the profitable use of reservoirs.

In tables which follow the writer has endeavored to compute the

value of reservoirs used in connection with steam. The several elements of this computation are :

First.—The quantity of water to be expected month by month and year by year from a given extent of drainage ground.

Second.—The quantity of steam-power required to make up the deficiencies of the water-power and maintain a constant total of power.

Third.—The change in the relative proportions of water and steam consequent upon reservoirs of different sizes.

Fourth.—The cost of steam-power.

The saving in steam-power that can be effected by reservoirs determines the amount that can be judiciously expended in their construction.

The flow from a drainage area of given extent has been observed daily by the engineers of the Boston Water Works for the last 30 years, viz., from 1863 to 1892, inclusive. These observations, previous to 1875, refer to the Cochituate drainage ground of some 16 sq. miles extent. Since that time they include the Sudbury Basin of some 75 sq. miles. These results may be applied, without serious risk of error, to any drainage area in New England.

Assume an establishment on a stream of 500 sq. miles drainage area, with a fall of 15 ft. and a requirement of 1 000 H. P., being furnished to that extent, both with water wheels and steam engines. To avoid the complication of pondage, assume the use of water to be continuous during the 24 hours. One thousand horse-power on a fall of 15 ft. requires about 800 cu. ft. per second. This is assumed as equivalent to 2 000 000 000 cu. ft. per month, which almost amounts to the supposition that the water runs to waste on Sundays and holidays.

Table No. 1 exhibits the flow of water in the natural condition of the stream, being the monthly average in cubic feet per second for the entire period of 30 years.

Table No. 2 gives the aggregate volume of water furnished each month in millions of cubic feet.

These two tables show in a marked manner the enormous fluctuations of the stream. The month of March, 1877, furnished 9 973 000 000 cu. ft.; September, only 119 000 000; the former being 3 723 cu. ft. per second; the latter, only 46. During March, assuming the flow to be uniform, 30 48-in. Rodney Hunt water wheels might have run at full gate. In September, a single wheel could hardly

have run at half gate. Other years show equally striking variations in other respects. Thus, in 1877, the highest monthly average exceeds the lowest 80 times. In 1889, the highest monthly average is only a little more than 4 times the lowest.

Table No. 3 gives the aggregate steam-power required each month to supplement the water-power, and make up a constant total of 1 000 H. P. The figures represent horse-powers 24 hours a day throughout the month.

Table No. 4 shows the effect of reservoirs of different sizes upon the quantity of water available for power. It shows, in millions of cubic feet, the volumes of water drawn from the reservoir, used and wasted, during the month, and that remaining in reservoir at the close of the month, for reservoirs of 3 000 000 000, 5 000 000 000 and 10 000 000 000 cu. ft., respectively. This table runs through the whole period of 30 years, month by month, and is necessarily voluminous.

Tables Nos. 5, 6 and 7 show, month by month, the steam-power required with reservoirs of 3 000 000 000, 5 000 000 000 and 10 000 000 000 cu. ft., respectively, expressed in Table No. 3.

Table No. 8 is a summary of the whole matter, being the cost of the steam-power each year. The first column contains the year; the second, the cost of steam-power in the natural condition of the stream; the third, the cost with reservoirs of 3 000 000 000 cu. ft.; the fourth, with 5 000 000 000; the fifth, with 10 000 000 000.

Cost of Steam-Power.—Charles E. Emery, M. Am. Soc. C. E., an engineer specially familiar with steam-power, in a carefully prepared paper presented to the Society in 1883, estimates the running cost of steam-power in engines of 500 H. P., working 10 hours a day, at $25 or $26 per annum per horse-power, the engine being supposed to run throughout the year at full load. In this estimate he puts the cost of coal at $4.17 per ton of 2 240 lbs., delivered in the coal house. This is too low for the inland towns of New England. The writer puts the cost of coal for the present purpose at $5 per ton of 2 000 lbs., delivered in coal house. This would raise the figure to $31 or $32 per annum per horse-power, or, we will say, for convenience, 10 cents per working day, or one cent per hour. This, it must be borne in mind, is for an engine running constantly at full load. An engine used to make up deficiencies of water-power, works under much less favorable conditions. It stops and starts with little notice. It works under loads varying

from a few horse-power up to nearly full capacity; most of the time, much below full capacity. Suppose, for instance, an engine of 500 H. P. to use, when running at full or nearly full load, 3 lbs. of coal per horse-power per hour. This engine, running without any load, would show, by indicator, some 50 H. P. In running to supply a deficiency of 50 H. P. it would indicate something over 100 H. P., and require 6 lbs. of coal per hour for each horse-power furnished. Moreover, wages of employees have to be paid, or their services employed in more or less unprofitable ways, while the engine is not running. For these reasons it appears that the power furnished by an engine under these conditions would cost 50% more than by one working constantly at full load. A part of this increase, however, is expense that attaches to the steam plant whether running or not, and would not appear in the sums credited to the reservoirs. The cost of the power has, therefore, been computed at 1¼ cents per hour, or 30 cents per day of 24 hours, per horse-power.

TABLE No. 1.

FLOW IN CUBIC FEET PER SECOND FROM 500 SQ. MILES OF NEW ENGLAND DRAINAGE AREA. AVERAGE FOR EACH MONTH.

	Jan.	Feb.	March.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1863.....	837	1 493	1 609	1 981	624	300	1 288	654	439	572	1 187	941
1864.....	1 036	725	1 756	1 187	702	219	178	281	220	620	560	577
1865.....	932	835	2 021	1 210	2 038	149	199	204	201	303	448	490
1866.....	316	1 363	763	730	559	493	520	278	600	403	443	676
1867.....	477	2 516	1 518	1 286	954	291	256	911	139	442	493	485
1868.....	529	519	1 665	1 559	2 676	712	195	512	848	412	878	507
1869.....	789	883	1 431	1 116	954	479	321	257	493	1 028	582	1 374
1870.....	2 042	1 887	1 466	3 079	720	434	230	178	385	481	394	334
1871.....	446	1 094	1 097	708	867	390	186	366	175	298	582	526
1872.....	498	432	611	1 380	477	667	60	572	762	733	896	524
1873.....	1 340	754	1 687	2 729	1 153	201	269	607	349	884	833	1 162
1874.....	1 539	1 051	798	1 429	1 205	878	412	399	237	225	260	221
1875.....	80	1 175	1 241	2 358	919	672	248	305	160	499	1 079	451
1876.....	497	1 058	3 431	2 547	881	171	141	313	142	181	841	351
1877.....	508	734	3 723	1 851	1 074	462	156	93	46	488	1 096	997
1878.....	1 400	1 907	2 713	1 258	1 078	391	99	367	124	399	1 306	2 458
1879.....	506	1 323	1 802	2 410	861	319	122	305	109	54	159	358
1880.....	867	1 382	1 060	904	397	136	136	92	62	78	158	135
1881.....	320	1 224	3 097	1 196	746	1 035	214	114	152	143	305	600
1882.....	959	1 859	2 196	670	1 000	409	67	43	237	231	167	243
1883.....	259	799	1 246	1 044	725	232	89	607	70	143	158	149
1884.....	770	2 198	2 928	2 222	797	322	173	198	34	64	135	715
1885.....	955	1 047	1 216	1 404	1 033	329	48	186	93	259	911	908
1886.....	1 131	3 713	1 592	1 506	557	157	89	73	91	112	520	789
1887.....	2 003	2 187	2 219	2 026	789	320	88	165	85	147	285	497
1888.....	813	1 509	2 504	2 046	1 263	326	93	293	893	1 546	2 133	2 354
1889.....	2 152	925	1 010	1 090	680	505	490	1 107	637	951	1 501	1 733
1890.....	970	1 182	2 818	1 450	1 057	438	83	102	354	1 757	940	770
1891.....	2 334	2 696	3 445	1 816	440	320	115	125	157	162	235	421
1892.....	1 446	716	1 512	674	973	331	165	217	177	97	544	375
Averages.	955	1 373	1 872	1 562	940	403	224	330	282	457	668	737

TABLE No. 2.

Flow From 500 Sq. Miles of New England Drainage Area. Aggregate for Each Month in Millions of Cubic Feet.

	Jan.	Feb.	March.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1863.....	2 242	3 612	4 309	5 134	1 672	778	3 450	1 754	1 138	1 533	3 078	2 520
1864.....	2 776	1 812	4 704	3 078	1 882	569	476	754	569	1 661	1 452	1 545
1865.....	2 497	2 021	5 413	3 137	5 459	395	534	546	522	813	1 161	1 312
1866.....	848	3 299	2 044	1 893	1 498	1 267	1 394	743	1 556	1 080	1 150	1 812
1867.....	1 277	6 086	4 065	3 334	2 555	755	685	2 439	360	1 185	1 277	1 301
1868.....	1 417	1 301	4 460	4 042	7 167	1 847	522	1 370	2 149	1 103	2 276	1 359
1869.....	2 114	2 137	3 833	2 892	2 555	1 243	859	673	1 277	2 753	1 510	3 682
1870.....	5 471	4 565	3 926	7 980	1 928	1 126	615	476	999	1 289	1 022	894
1871.....	1 196	2 648	2 939	1 835	2 323	1 010	499	987	453	801	1 510	1 409
1872.....	1 336	1 080	1 638	3 577	1 277	1 731	162	1 533	1 974	1 963	2 323	1 405
1873.....	3 589	1 823	4 518	7 074	3 090	522	720	1 626	906	2 369	2 160	3 113
1874.....	4 123	2 544	2 137	3 705	3 229	2 276	1 103	1 068	615	604	673	592
1875.....	213	2 800	3 324	6 113	2 461	1 743	665	820	416	1 338	2 611	1 209
1876.....	1 332	2 651	9 189	6 602	2 359	445	378	840	369	484	2 181	939
1877.....	1 363	1 776	9 973	4 799	2 883	1 197	418	251	119	1 309	2 842	2 671
1878.....	3 749	4 614	7 267	3 260	2 889	1 014	266	985	322	1 070	3 394	6 583
1879.....	1 451	3 201	4 827	6 248	2 308	828	326	819	282	146	412	958
1880.....	2 323	3 464	2 847	2 343	1 065	352	366	246	160	210	411	362
1881.....	859	2 893	8 296	3 100	1 999	2 682	572	306	395	384	792	1 606
1882	2 570	4 497	5 882	1 738	2 676	1 060	179	115	614	619	420	651
1883.....	693	1 933	3 337	2 706	1 943	601	239	1 626	182	384	411	400
1884.....	2 062	5 508	7 843	5 721	2 135	835	463	532	88	172	351	1 916
1885.....	2 559	2 533	3 258	3 639	2 768	854	129	498	243	696	2 361	2 342
1886.....	3 027	8 984	4 265	3 904	1 492	406	239	195	236	302	1 348	2 113
1887	5 365	5 291	5 942	5 253	2 089	829	237	443	222	393	739	1 332
1888.....	2 181	3 781	6 708	5 304	3 382	845	243	786	2 316	4 142	5 530	6 305
1889.....	5 765	2 237	2 774	2 827	1 822	1 310	1 312	2 966	1 652	2 548	3 892	4 643
1890.....	2 598	2 861	7 548	3 759	2 831	1 138	222	273	917	4 708	2 436	2 063
1891.....	6 253	6 523	9 227	4 806	1 207	829	309	337	406	435	611	1 128
1892.....	3 874	1 793	4 051	1 747	2 608	858	443	581	460	260	1 398	1 005
Averages.	2 571	3 342	5 018	4 052	2 518	1 045	601	886	730	1 225	1 724	1 974

TABLE No. 3.

STEAM-POWER REQUIRED TO MAKE UP A UNIFORM TOTAL OF 1 000 H. P. 24 HOURS A DAY IN A MILL COMMANDING 500 SQ. MILES OF NEW ENGLAND DRAINAGE AREA. WITHOUT RESERVOIRS. HEAD 15 FT. POWER EXPRESSED IN HORSE-POWER 24 HOURS A DAY.

	Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1863	.....	.....	.....	.....	164	611	.....	123	431	233	.....	.....
1864	.....	94	.....	.....	59	716	762	623	716	170	274	228
1865	.....	.....	.....	.....	.....	803	733	727	739	594	420	344
1866	576	.....	.....	54	251	367	303	629	222	460	425	94
1867	362	.....	.....	.....	.....	623	658	.....	820	408	362	350
1868	292	350	.....	.....	.....	77	739	315	.....	449	.....	321
1869	.....	.....	.....	.....	.....	379	571	664	362	.....	245	.....
1870	.....	.....	.....	.....	36	436	693	762	500	356	489	553
1871	402	.....	.....	83	.....	495	751	507	774	600	245	296
1872	332	460	181	.....	362	135	919	234	13	19	.....	298
1873	.....	89	.....	.....	.....	739	640	187	547	.....	.....	.....
1874	.....	.....	.....	.....	.....	.....	449	466	693	698	664	704
1875	894	.....	.....	.....	.....	129	668	590	792	331	.....	396
1876	334	.....	.....	.....	.....	778	811	580	816	758	.....	531
1877	319	112	.....	.....	.....	402	791	875	941	346	.....	.....
1878	.....	.....	.....	.....	.....	493	867	508	839	465	.....	.....
1879	275	.....	.....	.....	.....	586	837	591	859	927	794	521
1880	.....	.....	.....	.....	468	824	817	877	920	895	795	819
1881	571	.....	.....	.....	.....	.....	714	847	803	808	604	197
1882	.....	.....	.....	131	.....	470	911	943	693	691	790	675
1883	654	34	.....	.....	29	700	881	187	909	808	795	800
1884	.....	.....	.....	.....	.....	583	769	734	956	914	825	42
1885	.....	.....	.....	.....	.....	573	936	751	879	652	.....	.....
1886	.....	.....	.....	.....	254	797	881	903	882	849	326	.....
1887	.....	.....	.....	.....	.....	586	882	779	889	804	631	334
1888	.....	.....	.....	.....	.....	578	879	607	.....	.....	.....	.....
1889	.....	.....	.....	.....	89	345	344	.....	174	.....	.....	.....
1890	.....	.....	.....	.....	.....	431	889	864	542	.....	.....	.....
1891	.....	.....	.....	.....	397	586	846	832	797	783	695	436
1892	.....	104	.....	127	.....	571	779	710	770	870	301	498

TABLE No. 4.

EFFECT OF RESERVOIRS OF DIFFERENT SIZES UPON THE FLOW OF 500 SQ. MILES OF NEW ENGLAND DRAINAGE AREA, FROM 1863 TO 1892. QUANTITIES IN MILLIONS OF CUBIC FEET.

CAPACITY OF RESERVOIR.	3 000 000 000.				5 000 000 000.				10 000 000 000.			
	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.
1863. January	.....	2 000	.....	242	.....	2 000	.....	242	.....	2 000	.....	242
February	.....	2 000	.....	1 854	.....	2 000	.....	1 854	.....	2 000	.....	1 854
March	.....	2 000	1 163	3 000	.....	2 000	.....	4 163	.....	2 000	....	4 163
April	.....	2 000	3 134	3 000	.....	2 000	2 297	5 000	.....	2 000	.....	7 297
May	328	2 000	.....	2 672	328	2 000	.....	4 672	328	2 000	.....	6 969
June	1 222	2 000	.....	1 450	1 222	2 000	.....	3 450	1 222	2 000	.....	5 747
July	.....	2 000	.....	2 900	.....	2 000	.....	4 900	.....	2 000	.....	7 197
August	246	2 000	.....	2 654	246	2 000	.....	4 654	246	2 000	.....	6 951
September	862	2 000	.....	1 792	862	2 000	.....	3 792	862	2 000	.....	6 089
October	467	2 000	.....	1 325	467	2 000	.....	3 325	467	2 000	.....	5 622
November	.....	2 000	.....	2 403	.....	2 000	.....	4 403	.....	2 000	.....	6 700
December	.....	2 000	.....	2 923	.....	2 000	.....	4 923	.....	2 000	....	7 220
1864. January	.....	2 000	699	3 000	.....	2 000	699	5 000	.....	2 000	.....	7 996
February	188	2 000	.....	2 812	188	2 000	.....	4 812	188	2 000	.....	7 808
March	.....	2 000	2 516	3 000	.....	2 000	2 516	5 000	.....	2 000	512	10 000
April	.....	2 000	1 078	3 000	.....	2 000	1 078	5 000	.....	2 000	1 078	10 000
May	118	2 000	.....	2 882	118	2 000	.....	4 882	118	2 000	.....	9 882
June	1 431	2 000	.....	1 451	1 431	2 000	.....	3 451	1 431	2 000	.....	8 451
July	1 451	1 927	.....	0	1 524	2 000	.....	1 927	1 524	2 000	.....	6 927
August	.....	754	.....	0	1 246	2 000	.....	681	1 246	2 000	.....	5 681
September	....	569	.....	0	681	1 250	.....	0	1 431	2 000	.....	4 250
October	.....	1 661	.....	0	.....	1 661	.....	0	339	2 000	.....	3 911
November	.....	1 452	.....	0	.....	1 452	.....	0	548	2 000	.....	3 363
December	.....	1 545	.....	0	.....	1 545	.....	0	455	2 000	.....	2 908
1865. January	.....	2 000	.....	497	.....	2 000	.....	497	.....	2 000	.....	3 405
February	.....	2 000	.....	518	.....	2 000	.....	518	.....	2 000	.....	3 426
March	.....	2 000	931	3 000	.....	2 000	.....	3 931	.....	2 000	.....	6 639
April	.....	2 000	1 137	3 000	.....	2 000	68	5 000	.....	2 000	....	7 776
May	.....	2 000	3 459	3 000	.....	2 000	3 459	5 000	.....	2 000	1 235	10 000
June	1 605	2 000	.....	1 395	1 605	2 000	.....	3 395	1 605	2 000	.....	8 395
July	1 395	1 929	.....	0	1 466	2 000	.....	1 929	1 466	2 000	.....	6 929
August	....	546	.....	0	1 454	2 000	.....	475	1 454	2 000	.....	5 475
September	.....	522	.....	0	475	997	.....	0	1 478	2 000	.....	3 997
October	.....	813	.....	0	.....	813	.....	0	1 187	2 000	.....	2 810
November	.....	1 161	.....	0	.....	1 161	.....	0	839	2 000	....	1 971
December	.....	1 312	.....	0	.....	1 312	.....	0	688	2 000	.....	1 283
1866. January	.....	848	.....	0	.....	848	.....	0	1 152	2 000	.....	131
February	.....	2 000	.....	1 299	.....	2 000	.....	1 299	.....	2 000	.....	1 430
March	.....	2 000	.....	1 343	.....	2 000	.....	1 343	.....	2 000	.....	1 474
April	107	2 000	.....	1 236	107	2 000	.....	1 236	107	2 000	.....	1 367
May	502	2 000	.....	734	502	2 000	.....	734	502	2 000	.....	865
June	733	2 000	.....	1	733	2 000	.....	1	733	2 000	.....	132
July	1	1 395	.....	0	1	1 395	.....	0	132	1 526	.....	0
August	.....	743	.....	0	.....	743	.....	0	.....	743	.....	0
September	.....	1 556	.....	0	.....	1 556	.....	0	.....	1 556	.....	0
October	.....	1 080	.....	0	.....	1 080	.....	0	.....	1 080	.....	0
November	.....	1 150	.....	0	.....	1 150	.....	0	....	1 150	.....	0
December	....	1 812	.....	0	....	1 812	.....	0	.....	1 812	.....	0

TABLE No. 4—(*Continued*).

Capacity of Reservoir.	3 000 000 000.				5 000 000 000.				10 000 000 000.			
	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.
1867. January	.....	1 277	.....	0	.....	1 277	.....	0	.....	1 277	.....	0
February	.....	2 000	1 086	3 000	.....	2 000	.....	4 086	.....	2 000	.....	4 086
March	.....	2 000	2 065	3 000	.....	2 000	1 151	5 000	.....	2 000	.....	6 151
April	.....	2 000	1 334	3 000	.....	2 000	1 334	5 000	.....	2 000	.....	7 485
May	.....	2 000	555	3 000	.....	2 000	555	5 000	.....	2 000	.....	8 040
June	1 245	2 000	.....	1 755	1 245	2 000	.....	3 755	1 245	2 000	.....	6 795
July	1 315	2 000	.....	440	1 315	2 000	.....	2 440	1 315	2 000	.....	5 480
August	.....	2 000	.....	879	.....	2 000	.....	2 879	.....	2 000	.....	5 919
September	879	1 239	.....	0	1 640	2 000	.....	1 239	1 640	2 000	.....	4 279
October	.....	1 185	.....	0	815	2 000	.....	424	815	2 000	.....	3 464
November	.....	1 277	.....	0	424	1 701	.....	0	723	2 000	.....	2 741
December	.....	1 301	.....	0	.....	1 301	.....	0	699	2 000	.....	2 042
1868. January	.....	1 417	.....	0	.....	1 417	.....	0	583	2 000	.....	1 459
February	.....	1 301	.....	0	.....	1 301	.....	0	699	2 000	.....	760
March	.....	2 000	.....	2 460	.....	2 000	.....	2 460	.....	2 000	.....	3 220
April	.....	2 000	1 502	3 000	.....	2 000	.....	4 502	.....	2 000	.....	5 262
May	.....	2 000	5 167	3 000	.....	2 000	4 669	5 000	.....	2 000	429	10 000
June	153	2 000	.....	2 847	153	2 000	.....	4 847	153	2 000	.....	9 847
July	1 478	2 000	.....	1 369	1 478	2 000	.....	3 369	1 478	2 000	.....	8 369
August	630	2 000	.....	739	630	2 000	.....	2 739	630	2 000	.....	7 739
September	.....	2 000	.....	888	.....	2 000	.....	2 888	.....	2 000	.....	7 888
October	888	1 991	.....	0	897	2 000	.....	1 991	897	2 000	.....	6 991
November	.....	2 000	.....	276	.....	2 000	.....	2 267	.....	2 000	.....	7 267
December	276	1 635	.....	0	641	2 000	.....	1 626	641	2 000	.....	6 626
1869. January	.....	2 000	.....	114	.....	2 000	.....	1 740	.....	2 000	.....	6 740
February	.....	2 000	.....	251	.....	2 000	.....	1 877	.....	2 000	.....	6 877
March	.....	2 000	.....	2 084	.....	2 000	.....	3 710	.....	2 000	.....	8 710
April	.....	2 000	.....	2 976	.....	2 000	.....	4 602	.....	2 000	.....	9 602
May	.....	2 000	531	3 000	.....	2 000	157	5 000	.....	2 000	157	10 000
June	757	2 000	.....	2 243	757	2 000	.....	4 243	757	2 000	.....	9 243
July	1 141	2 000	.....	1 102	1 141	2 000	.....	3 102	1 141	2 000	.....	8 102
August	1 102	1 675	.....	0	1 327	2 000	.....	1 775	1 327	2 000	.....	6 775
September	.....	1 277	.....	0	723	2 000	.....	1 052	723	2 000	.....	6 052
October	.....	2 000	.....	753	.....	2 000	.....	1 805	.....	2 000	.....	6 805
November	490	2 000	.....	263	.....	2 000	.....	1 315	490	2 000	.....	6 315
December	.....	2 000	.....	1 945	.....	2 000	.....	2 997	.....	2 000	.....	7 997
1870. January	.....	2 000	2 416	3 000	.....	2 000	1 468	5 000	.....	2 000	1 468	10 000
February	.....	2 000	2 565	3 000	.....	2 000	2 565	5 000	.....	2 000	2 565	10 000
March	.....	2 000	1 926	3 000	.....	2 000	1 926	5 000	.....	2 000	1 926	10 000
April	.....	2 000	5 980	3 000	.....	2 000	5 980	5 000	.....	2 000	5 980	10 000
May	72	2 000	.....	2 928	72	2 000	.....	4 928	72	2 000	.....	9 928
June	874	2 000	.....	2 054	874	2 000	.....	4 054	874	2 000	.....	9 054
July	1 385	2 000	.....	669	1 385	2 000	.....	2 669	1 385	2 000	.....	7 679
August	669	1 145	.....	0	1 524	2 000	.....	1 145	1 524	2 000	.....	6 145
September	.....	999	.....	0	1 001	2 000	.....	144	1 001	2 000	.....	5 144
October	.....	1 289	.....	0	144	1 433	.....	0	711	2 000	.....	4 435
November	.....	1 022	.....	0	.....	1 022	.....	0	978	2 000	.....	3 455
December	.....	894	.....	0	.....	894	.....	0	1 106	2 000	.....	2 349
1871. January	.....	1 196	.....	0	.....	1 196	.....	0	804	2 000	.....	1 545
February	.....	2 000	.....	648	.....	2 000	.....	648	.....	2 000	.....	2 193
March	.....	2 000	.....	1 587	.....	2 000	.....	1 587	.....	2 000	.....	3 132
April	165	2 000	.....	1 422	165	2 000	.....	1 422	165	2 000	.....	2 967
May	.....	2 000	.....	1 745	.....	2 000	.....	1 745	.....	2 000	.....	3 290
June	990	2 000	.....	755	990	2 000	.....	755	990	2 000	.....	2 300
July	755	1 254	.....	0	755	1 254	.....	0	1 501	2 000	.....	799
August	.....	987	.....	0	.....	987	.....	0	799	1 786	.....	0

TABLE No. 4—(*Continued*).

Capacity of Reservoir.	3 000 000 000.				5 000 000 000.				10 000 000 000.			
	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.
1871. September	.....	453	.....	0	.....	453	.....	0	.....	453	.....	0
October	....	801	.....	0	.....	801	.....	0	.....	801	.....	0
November	.....	1 510	.....	0	.....	1 510	.....	0	.....	1 510	.....	0
December	.....	1 409	.....	0	.....	1 409	.....	0	....	1 409	.....	0
1872. January	.....	1 336	.....	0	.....	1 332	.....	0	.....	1 336	.....	0
February	.....	1 080	.....	0	.....	1 080	.....	0	.....	1 080	.....	0
March	.....	1 638	.....	0	.....	1 638	.....	0	.....	1 638	.....	0
April	.....	2 000	.....	1 577	.....	2 000	.....	1 577	.....	2 000	.....	1 577
May	723	2 000	.....	854	723	2 000	.....	854	723	2 000	.....	854
June	269	2 000	.....	585	269	2 000	.....	585	269	2 000	.....	585
July	585	747	.....	0	585	747	.....	0	585	747	.....	0
August	.....	1 533	.....	0	.....	1 533	.....	0	.....	1 533	.....	0
September	.....	1 974	.....	9	.....	1 974	.....	0	.....	1 974	.....	0
October	.....	1 963	.....	0	.....	1 963	.....	0	.....	1 963	.....	0
November	.....	2 000	.....	323	.....	2 000	.....	323	.....	2 000	.....	323
December	323	1 728	.....	0	323	1 728	.....	0	323	1 728	.....	0
1873. January	.....	2 000	.....	1 589	.....	2 000	.....	1 589	.....	2 000	.....	1 589
February	177	2 000	.....	1 412	177	2 000	.....	1 412	177	2 000	.....	1 412
March	.....	2 000	930	3 000	.....	2 000	.....	3 930	.....	2 000	.....	3 930
April	.....	2 000	5 074	3 000	.....	2 000	4 004	5 000	.....	2 000	.....	9 004
May	.....	2 000	1 090	3 000	.....	2 000	1 090	5 000	.....	2 000	94	10 000
June	1 478	2 000	.....	1 522	1 478	2 000	.....	3 522	1 478	2 000	.....	8 522
July	1 280	2 000	.....	242	1 280	2 000	.....	2 242	1 280	2 000	.....	7 242
August	.....	1 868	.....	0	374	2 000	.....	1 868	374	2 000	.....	6 868
September	.....	906	.....	0	1 094	2 000	.....	774	1 094	2 000	.....	5 774
October	.....	2 000	.....	369	.....	2 000	.....	1 143	.....	2 000	.....	6 143
November	.....	2 000	.....	529	.....	2 000	.....	1 303	.....	2 000	.....	6 303
December	.....	2 000	.....	1 642	.....	2 000	.....	2 416	.....	2 000	.....	7 416
1874. January	.....	2 000	765	3 000	.....	2 000	.....	4 539	.....	2 000	.....	9 539
February	.....	2 000	544	3 000	.....	2 000	83	5 000	.....	2 000	83	10 000
March	.....	2 000	137	3 000	.....	2 000	137	5 000	.....	2 000	137	10 000
April	.....	2 000	1 705	3 000	.....	2 000	1 705	5 000	.....	2 000	1 705	10 000
May	.....	2 000	1 229	3 000	.....	2 000	1 229	5 000	.....	2 000	1 229	10 000
June	.....	2 000	276	3 000	.....	2 000	276	5 000	.....	2 000	276	10 000
July	897	2 000	.....	2 103	897	2 000	.....	4 103	897	2 000	.....	9 103
August	932	2 000	.....	1 171	932	2 000	.....	3 171	932	2 000	.....	8 171
September	1 171	1 786	.....	0	1 385	2 000	.....	1 786	1 385	2 000	.....	6 786
October	.....	604	.....	0	1 396	2 000	.....	390	1 396	2 000	.....	5 390
November	.....	673	.....	0	390	1 063	.....	0	1 327	2 000	.....	4 063
December	.....	592	.....	0	.....	592	.....	0	1 408	2 000	.....	2 655
1875. January	.....	213	.....	0	.....	213	.....	0	.....	2 000	.....	868
February	.....	2 000	.....	800	.....	2 000	.....	800	.....	2 000	.....	1 668
March	.....	2 000	.....	2 124	.....	2 000	.....	2 124	.....	2 000	.....	2 992
April	.....	2 000	3 237	3 000	.....	2 000	1 237	5 000	.....	2 000	.....	7 105
May	.....	2 000	461	3 000	.....	2 000	461	5 000	.....	2 000	.....	7 556
June	257	2 000	.....	2 743	257	2 000	.....	4 743	257	2 000	.....	7 309
July	1 335	2 000	.....	1 408	1 335	2 000	.....	3 408	1 335	2 000	.....	5 974
August	1 180	2 000	.....	228	1 180	2 000	.....	2 228	1 180	2 000	.....	4 794
September	228	644	.....	0	1 584	2 000	.....	644	1 584	2 000	.....	3 210
October	.....	1 338	.....	0	644	1 982	.....	0	662	2 000	.....	2 548
November	.....	2 000	.....	611	.....	2 000	.....	611	.....	2 000	.....	3 159
December	611	1 820	.....	0	611	1 820	.....	0	791	2 000	....	2 368
1876. January	.....	1 332	.....	0	.....	1 332	.....	0	668	2 000	.....	1 700
February	.....	2 000	.....	651	.....	2 000	.....	651	....	2 000	.....	2 351
March	.....	2 000	4 840	3 000	.....	2 000	2 840	5 000	.....	2 000	.....	9 540

TABLE No. 4—(*Continued*).

Capacity of Reservoir.	3 000 000 000.				5 000 000 000.				10 000 000 000.			
	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.
1876. April	.....	2 000	4 602	3 000	.....	2 000	4 602	5 000	.....	2 000	4 142	10 000
May	.....	2 000	359	3 000	.....	2 000	359	5 000	.....	2 000	359	10 000
June	1 555	2 000	.....	1 445	1 555	2 000	.....	3 445	1 555	2 000	.....	8 445
July	1 445	1 823	.....	0	1 622	2 000	.....	1 823	1 622	2 000	.....	6 823
August	.....	840	.....	0	1 160	2 000	.....	663	1 160	2 000	.....	5 663
September	.....	369	.....	0	663	1 032	.....	0	1 631	2 000	.....	4 032
October	.....	484	.....	0	.....	484	.....	0	1 516	2 000	.....	2 516
November	.....	2 000	.....	181	.....	2 000	.....	181	.....	2 000	.....	2 697
December	181	1 120	.....	0	181	1 120	.....	0	1 061	2 000	.....	1 636
1877. January	.....	1 363	.....	0	.....	1 363	.....	0	637	2 000	.....	999
February	.....	1 776	.....	0	.....	1 776	.....	0	224	2 000	.....	775
March	.....	2 000	4 973	3 000	.....	2 000	2 973	5 000	.....	2 000	.....	8 748
April	.....	2 000	2 799	3 000	.....	2 000	2 799	5 000	.....	2 000	1 547	10 000
May	.....	2 000	883	3 000	.....	2 000	883	5 000	.....	2 000	883	10 000
June	803	2 000	.....	2 197	803	2 000	.....	4 197	803	2 000	.....	9 197
July	1 582	2 000	.....	615	1 582	2 000	.....	2 615	1 582	2 000	.....	7 615
August	615	1 366	.....	0	1 749	2 000	.....	866	1 749	2 000	.....	5 866
September	.....	119	.....	0	866	985	.....	0	1 881	2 000	.....	3 985
October	.....	1 309	.....	0	.....	1 309	.....	0	691	2 000	.....	3 294
November	.....	2 000	.....	842	.....	2 000	.....	842	.....	2 000	.....	4 136
December	.....	2 000	.....	1 513	.....	2 000	.....	1 513	.....	2 000	.....	4 807
1878. January	.....	2 000	262	3 000	.....	2 000	.....	3 262	.....	2 000	.....	6 556
February	.....	2 000	2 614	3 000	.....	2 000	876	5 000	.....	2 000	.....	9 170
March	.....	2 000	5 267	3 000	.....	2 000	5 267	5 000	.....	2 000	4 437	10 000
April	.....	2 000	1 260	3 000	.....	2 000	1 260	5 000	.....	2 000	1 260	10 000
May	.....	2 000	889	3 000	.....	2 000	889	5 000	.....	2 000	889	10 000
June	986	2 000	.....	2 014	986	2 000	.....	4 014	986	2 000	....	9 014
July	1 734	2 000	.....	280	1 734	2 000	.....	2 280	1 734	2 000	....	7 280
August	280	1 265	.....	0	1 015	2 000	.....	1 265	1 015	2 000	.....	6 265
September	....	322	.....	0	1 265	1 587	.....	0	1 678	2 000	.....	4 587
October	.....	1 070	.....	0	.....	1 070	.....	0	930	2 000	.....	3 657
November	....	2 000	.....	1 394	.....	2 000	.....	1 394	.....	2 000	.....	5 051
December	.....	2 000	2 977	3 000	.....	2 000	977	5 000	.....	2 000	.....	9 634
1879. January	549	2 000	.....	2 451	549	2 000	.....	4 451	549	2 000	.....	9 085
February	.....	2 000	652	3 000	.....	2 000	652	5 000	.....	2 000	286	10 000
March	.....	2 000	2 827	3 000	.....	2 000	2 827	5 000	.....	2 000	2 827	10 000
April	.....	2 000	4 248	3 000	.....	2 000	4 248	5 000	.....	2 000	4 248	10 000
May	.....	2 000	308	3 000	.....	2 000	308	5 000	.....	2 000	308	10 000
June	1 172	2 000	.....	1 828	1 172	2 000	.....	3 828	1 172	2 000	.....	8 828
July	1 674	2 000	.....	154	1 674	2 000	.....	2 154	1 674	2 000	.....	7 154
August	154	973	.....	0	1 181	2 000	.....	973	1 181	2 000	.....	5 973
September	.....	282	.....	0	973	1 255	.....	0	1 718	2 000	.....	4 255
October	.....	146	.....	0	.....	146	.....	0	1 854	2 000	.....	2 401
November	.....	412	.....	0	.....	412	.....	0	1 588	2 000	.....	813
December	.....	958	.....	0	.....	958	.....	0	813	1 771	.....	0
1880. January	.....	2 000	.....	323	.....	2 000	.....	323	.....	2 000	.....	323
February	.....	2 000	.....	1 787	.....	2 000	.....	1 787	.....	2 000	.....	1 787
March	.....	2 000	.....	2 634	.....	2 000	.....	2 634	.....	2 000	.....	2 634
April	.....	2 000	.....	2 977	.....	2 000	.....	2 977	.....	2 000	.....	2 977
May	935	2 000	.....	2 042	935	2 000	.....	2 042	935	2 000	.....	2 042
June	1 648	2 000	.....	394	1 648	2 000	.....	394	1 648	2 000	.....	394
July	394	760	.....	0	394	760	.....	0	394	760	...	0
August	.....	246	.....	0	.....	246		0	.....	246	.....	0
September	.....	160	.....	0	.....	160	.....	0	.....	160	.....	0
October	.....	210	.....	0	.....	210	.....	0	.....	210	.....	0

TABLE No. 4—(*Continued*).

Capacity of Reservoir.	3 000 000 000.				5 000 000 000.				10 000 000 000.			
	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.
1880. November	.....	411	.....	0	.....	411	.....	0	.....	411	.....	0
December	.....	362	.....	0	.....	362	.....	0	.....	362	.....	0
1881. January	.....	859	.....	0	.....	859	.....	0	.....	859	.....	0
February	.....	2 000	.....	893	.....	2 000	.....	893	.....	2 000	.....	893
March	.....	2 000	4 189	3 000	.....	2 000	2 189	5 000	.....	2 000	.....	7 189
April	.....	2 000	1 100	3 000	.....	2 000	1 100	5 000	.....	2 000	.....	8 289
May	1	2 000	.....	2 999	1	2 000	.....	4 999	1	2 000	.....	8 288
June	....	2 000	681	3 000	.....	2 000	681	5 000	.....	2 000	.....	8 970
July	1 428	2 000	.....	1 572	1 428	2 000	.....	3 572	1 428	2 000	.....	7 542
August	1 572	1 878	.....	0	1 694	2 000	....	1 878	1 694	2 000	.....	5 848
September	.....	395	.....	0	1 605	2 000	.....	273	1 605	2 000	.....	4 243
October	.....	384	.....	0	273	657	.....	0	1 616	2 000	.....	2 627
November	.....	792	.....	0	.....	792	.....	0	1 208	2 000	.....	1 419
December	.....	1 606	.....	0	.....	1 606	.....	0	394	2 000	.....	1 025
1882. January	.....	2 000	.....	570	.....	2 000	.....	570	.....	2 000	.....	1 595
February	.....	2 000	67	3 000	.....	2 000	.....	3 067	.....	2 000	.....	4 092
March	.....	2 000	3 882	3 000	.....	2 000	1 949	5 000	.....	2 000	.....	7 974
April	262	2 000	.....	2 738	262	2 000	.....	4 738	262	2 000	.....	7 712
May	.....	2 000	414	3 000	.....	2 000	414	5 000	.....	2 000	.....	8 388
June	940	2 000	.....	2 060	940	2 000	.....	4 060	940	2 000	.....	7 448
July	1 821	2 000	.....	239	1 821	2 000	.....	2 239	1 821	2 000	.....	5 627
August	239	354	.....	0	1 885	2 000	.....	354	1 885	2 000	.....	3 742
September	.....	614	.....	0	354	968	.....	0	1 386	2 000	.....	2 356
October	.....	619	.....	0	.....	619	.....	0	1 381	2 000	.....	975
November	.....	420	.....	0	.....	420	.....	0	975	1 395	.....	0
December	.....	651	.....	0	.....	651	.....	0	.....	651	.....	0
1883. January	.....	693	.....	0	.....	693	.....	0	.....	693	.....	0
February	.....	1 933	.....	0	.....	1 933	.....	0	.....	1 933	.....	0
March	.....	2 000	.....	1 337	.....	2 000	.....	1 337	.....	2 000	.....	1 337
April	.....	2 000	.....	2 043	.....	2 000	.....	2 043	.....	2 000	....	2 043
May	57	2 000	.....	1 986	57	2 000	....	1 986	57	2 000	.....	1 986
June	1 399	2 000	.....	587	1 399	2 000	.....	587	1 399	2 000	.....	587
July	587	826	.....	0	587	826	.....	0	587	826	.....	0
August	.....	1 626	.....	0	.....	1 626	.....	0	.....	1 626	.....	0
September	.....	182	.....	0	.....	182	....	0	.....	182	.....	0
October	.....	384	.....	0	.....	384	.....	0	.....	384	.....	0
November	.....	411	.....	0	.....	411	.....	0	....	411	.....	0
December	....	400	.....	0	.....	400	.....	0	.....	400	.....	0
1884. January	.....	2 000	.....	62	.....	2 000	.....	62	.....	2 000	.....	62
February	.....	2 000	570	3 000	.....	2 000	.....	3 570	.....	2 000	.....	3 570
March	...	2 000	5 843	3 000	.....	2 000	4 413	5 000	.....	2 000	.....	9 413
April	.....	2 000	3 721	3 000	.....	2 000	3 721	5 000	.....	2 000	3 134	10 000
May	.....	2 000	135	3 000	.....	2 000	135	5 000	.....	2 000	135	10 000
June	1 165	2 000	.....	1 835	1 165	2 000	.....	3 835	1 165	2 000	.....	8 835
July	1 537	2 000	.....	298	1 537	2 000	.....	2 298	1 537	2 000	.....	7 298
August	298	830	.....	0	1 468	2 000	.....	830	1 468	2 000	.....	5 830
September	.....	88	.....	0	830	918	.....	0	1 912	2 000	.....	3 918
October	.....	172	.....	0	.....	172	.....	0	1 828	2 000	.....	2 090
November	.....	351	.....	0	.....	351	.....	0	1 649	2 000	.....	441
December	.....	1 916	.....	0	.....	1 916	.....	0	84	2 000	.....	357
1885. January	.....	2 000	.....	559	.....	2 000	.....	559	.....	2 000	.....	916
February	.....	2 000	.....	1 092	.....	2 000	.....	1 092	.....	2 000	.....	1 449
March	.....	2 000	...	2 350	.....	2 000	.....	2 350	.....	2 000	.....	2 707
April	.....	2 000	989	3 000	.....	2 000	.....	3 989	.....	2 000	.....	4 346
May	.....	2 000	768	3 000	.....	2 000	.....	4 757	.....	2 000	.....	5 114

TABLE No. 4—(*Continued*).

Capacity of Reservoir.	3 000 000 000.				5 000 000 000.				10 000 000 000.			
	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.
1885. June	1 146	2 000	.....	1 854	1 146	2 000	.....	3 611	1 146	2 000	.....	3 968
July	1 854	1 983	.....	0	1 871	2 000	.....	1 740	1 871	2 000	.....	2 097
August	.....	498	.....	0	1 502	2 000	.....	238	1 502	2 000	.....	595
September	.....	243	.....	0	238	481	.....	0	595	838	.....	0
October	.....	696	.....	0	.....	696	.....	0	.....	696	.....	0
November	.....	2 000	.....	361	.....	2 000	.....	361	.....	2 000	.....	361
December	.....	2 000	.....	703	.....	2 000	.....	703	.....	2 000	.....	703
1886. January	.....	2 000	.....	1 730	.....	2 000	.....	1 730	.....	2 000	.....	1 730
February	.....	2 000	5 714	3 000	.....	2 000	3 714	5 000	.....	2 000	.....	8 714
March	.....	2 000	2 265	3 000	.....	2 000	2 265	5 000	.....	2 000	979	10 000
April	.....	2 000	1 904	3 000	.....	2 000	1 904	5 000	.....	2 000	1 904	10 000
May	508	2 000	.....	2 492	508	2 000	.....	4 492	508	2 000	.....	9 492
June	1 594	2 000	.....	898	1 594	2 000	.....	2 898	1 594	2 000	.....	7 898
July	898	1 137	.....	0	1 761	2 000	.....	1 137	1 761	2 000	.....	6 137
August	.....	195	.....	0	1 137	1 332	.....	0	1 805	2 000	.....	4 332
September	.....	236	.....	0	.....	236	.....	0	1 764	2 000	.....	2 568
October	.....	302	.....	0	.....	302	.....	0	1 698	2 000	.....	870
November	.....	1 348	.....	0	.....	1 348	.....	0	652	2 000	.....	218
December	.....	2 000	.....	113	.....	2 000	.....	113	.....	2 000	.....	331
1887. January	.....	2 000	478	3 000	.....	2 000	.....	3 478	.....	2 000	.....	3 696
February	.....	2 000	3 291	3 000	.....	2 000	1 769	5 000	.....	2 000	.....	6 987
March	.....	2 000	3 942	3 000	.....	2 000	3 942	5 000	.....	2 000	929	10 000
April	.....	2 000	3 253	3 000	.....	2 000	3 253	5 000	.....	2 000	3 253	10 000
May	.....	2 000	89	3 000	.....	2 000	83	5 000	.....	2 000	89	10 000
June	1 171	2 000	.....	1 829	1 171	2 000	.....	3 829	1 171	2 000	.....	8 829
July	1 763	2 000	.....	66	1 763	2 000	.....	2 066	1 763	2 000	.....	7 066
August	66	509	.....	0	1 557	2 000	.....	509	1 557	2 000	.....	5 509
September	.....	222	.....	0	509	731	.....	0	1 778	2 000	.....	3 731
October	.....	393	.....	0	.....	393	.....	0	1 607	2 000	.....	2 124
November	.....	739	.....	0	.....	739	.....	0	1 261	2 000	.....	863
December	.....	1 332	.....	0	.....	1 332	.....	0	668	2 000	.....	195
1888. January	.....	2 000	.....	181	.....	2 000	.....	181	.....	2 000	.....	376
February	.....	2 000	.....	1 962	.....	2 000	.....	1 962	.....	2 000	.....	2 157
March	.....	2 000	3 670	3 000	.....	2 000	1 670	5 000	.....	2 000	.....	6 865
April	.....	2 000	3 304	3 000	.....	2 000	3 304	5 000	.....	2 000	169	10 000
May	.....	2 000	1 382	3 000	.....	2 000	1 382	5 000	.....	2 000	1 382	10 000
June	1 155	2 000	.....	1 845	1 155	2 000	.....	3 845	1 155	2 000	.....	8 845
July	1 757	2 000	.....	88	1 757	2 000	.....	2 088	1 757	2 000	.....	7 088
August	88	874	.....	0	1 214	2 000	.....	874	1 214	2 000	.....	5 874
September	.....	2 000	.....	316	.....	2 000	.....	1 190	.....	2 000	.....	6 190
October	.....	2 000	.....	2 458	.....	2 000	.....	3 332	.....	2 000	.....	8 332
November	.....	2 000	2 988	3 000	.....	2 000	1 862	5 000	.....	2 000	1 862	10 000
December	.....	2 000	4 305	3 000	.....	2 000	4 305	5 000	.....	2 000	4 305	10 000
1889. January	.....	2 000	3 765	3 000	.....	2 000	3 765	5 000	.....	2 000	3 765	10 000
February	.....	2 000	237	3 000	.....	2 000	237	5 000	.....	2 000	237	10 000
March	.....	2 000	774	3 000	.....	2 000	774	5 000	.....	2 000	774	10 000
April	.....	2 000	827	3 000	.....	2 000	827	5 000	.....	2 000	827	10 000
May	178	2 000	.....	2 822	178	2 000	.....	4 822	178	2 000	.....	9 822
June	690	2 000	.....	2 132	690	2 000	.....	4 132	690	2 000	.....	9 132
July	688	2 000	.....	1 444	688	2 000	.....	3 444	688	2 000	.....	8 444
August	.....	2 000	.....	2 410	.....	2 000	.....	4 410	.....	2 000	.....	9 410
September	348	2 000	.....	2 062	348	2 000	.....	4 062	348	2 000	.....	9 062
October	.....	2 000	.....	2 610	.....	2 000	.....	4 610	.....	2 000	.....	9 610
November	.....	2 000	1 502	3 000	.....	2 000	1 502	5 000	.....	2 000	1 502	10 000
December	.....	2 000	2 643	3 000	.....	2 000	2 643	5 000	.....	2 000	2 643	10 000

TABLE No. 4—(*Continued*).

Capacity of Reservoir.	3 000 000 000.				5 000 000 000.				10 000 000 000.			
	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir at end of month.	Drawn from reservoir.	Used.	Wasted.	In reservoir end of month.
1890. January	.....	2 000	598	3 000	.....	2 000	598	5 000	.....	2 000	598	10 000
February	.....	2 000	861	3 000	.....	2 000	861	5 000	.....	2 000	861	10 000
March	.....	2 000	5 548	3 000	.....	2 000	5 548	5 000	.....	2 000	5 548	10 000
April	.....	2 000	1 759	3 000	.....	2 000	1 759	5 000	.....	2 000	1 759	10 000
May	.....	2 000	831	3 000	.....	2 000	831	5 000	.....	2 000	831	10 000
June	862	2 000	.....	2 138	862	2 000	.....	4 138	862	2 000	.....	9 138
July	1 778	2 000	.....	360	1 778	2 000	.....	2 360	1 778	2 000	.....	7 360
August	.....	633	.....	0	1 727	2 000	.....	633	1 727	2 000	.....	5 633
September	.....	917	.....	0	633	1 550	.....	0	1 083	2 000	.....	4 550
October	.....	2 000	.....	2 708	.....	2 000	.....	2 708	.....	2 000	.....	7 258
November	.....	2 000	144	3 000	.....	2 000	.....	3 144	.....	2 000	.....	7 694
December	.....	2 000	63	3 000	.....	2 000	.....	3 207	.....	2 000	.....	7 757
1891. January	.....	2 000	4 253	3 000	.....	2 000	2 460	5 000	.....	2 000	2 010	10 000
February	.....	2 000	4 523	3 000	.....	2 000	4 523	5 000	.....	2 000	4 523	10 000
March	.....	2 000	7 227	3 000	.....	2 000	7 227	5 000	.....	2 000	7 227	10 000
April	.....	2 000	2 806	3 000	.....	2 000	2 806	5 000	.....	2 000	2 806	10 000
May	793	2 000	.....	2 207	793	2 000	.....	4 207	793	2 000	.....	9 207
June	1 171	2 000	.....	1 036	1 171	2 000	.....	3 096	1 171	2 000	.....	8 036
July	1 036	1 347	.....	0	1 691	2 000	.....	1 405	1 691	2 000	.....	6 347
August	.....	337	.....	0	1 405	1 742	.....	0	1 663	2 000	.....	4 684
September	.....	406	.....	0	.....	406	.....	0	1 594	2 000	.....	3 090
October	.....	435	.....	0	.....	435	.....	0	1 565	2 000	.....	1 525
November	.....	611	.....	0	.....	611	.....	0	1 389	2 000	.....	136
December	.....	1 128	.....	0	.....	1 128	.....	0	136	2 000	.....	0
1892. January	.....	2 000	.....	1 874	.....	2 000	.....	1 874	.....	2 000	.....	1 874
February	207	2 000	.....	1 667	207	2 000	.....	1 667	207	2 000	.....	1 667
March	.....	2 000	718	3 000	.....	2 000	.....	3 718	.....	2 000	.....	3 718
April	253	2 000	.....	2 747	253	2 000	.....	3 465	253	2 000	.....	3 465
May	.....	2 000	355	3 000	.....	2 000	.....	4 073	.....	2 000	.....	4 073
June	1 142	2 000	.....	1 858	1 142	2 000	.....	2 931	1 142	2 000	.....	2 931
July	1 557	2 000	.....	301	1 557	2 000	.....	1 374	1 557	2 000	.....	1 374
August	301	882	.....	0	1 374	1 955	.....	0	1 374	1 955	.....	0
September	.....	460	.....	0	.....	460	.....	0	.....	460	.....	0
October	.....	260	.....	0	.....	260	.....	0	.....	260	.....	0
November	.....	1 398	.....	0	.....	1 398	.....	0	.....	1 398	.....	0
December	.....	1 050	.....	0	.....	1 050	.....	0	.....	1 050	.....	0

TABLE No. 5.

STEAM-POWER REQUIRED TO MAKE UP A UNIFORM TOTAL OF 1 000 H. P. 24 HOURS A DAY, IN A MILL COMMANDING 500 SQ. MILES OF NEW ENGLAND DRAINAGE AREA, WITH RESERVOIRS TO THE EXTENT OF 3 000 000 000 CU. FT. HEAD, 15 FT. POWER EXPRESSED IN HORSE-POWER 24 HOURS A DAY.

	Jan.	Feb.	March.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1863												
1864							36	623	716	170	274	228
1865							35	727	739	594	420	344
1866	576						303	628	222	460	425	94
1867	362								381	408	362	350
1868	292	350								4		182
1869								163	362			
1870								428	501	356	489	553
1871	402						373	507	774	600	245	296
1872	332	460	181				627	234	13	19		136
1873								66	547			
1874									107	698	664	704
1875	894								678	331		90
1876	334						89	580	816	758		440
1877	319	112						317	940	346		
1878								368	839	465		
1879								514	859	927	794	521
1880							620	877	920	895	795	819
1881	571							61	803	808	604	197
1882								823	693	691	790	675
1883	654	34					587	187	909	808	795	800
1884								585	956	914	825	42
1885							8	751	879	652		
1886							432	903	882	849	326	
1887								746	889	804	631	334
1888								563				
1889												
1890								684	542			
1891							327	832	797	783	695	436
1892								558	770	870	301	475

TABLE No. 6.

STEAM-POWER REQUIRED TO MAKE UP A UNIFORM TOTAL OF 1 000 H. P. 24 HOURS A DAY IN A MILL COMMANDING 500 SQ. MILES OF NEW ENGLAND DRAINAGE AREA, WITH RESERVOIRS TO THE EXTENT OF 5 000 000 000 CU. FT. HEAD, 15 FT. POWER EXPRESSED IN HORSE-POWER, 24 HOURS A DAY.

	Jan.	Feb.	March.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1863	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1864	.....	.....	.....	.....	.....	.....	.....	.....	375	170	274	228
1865	.....	.....	.....	.....	.....	.....	.....	.....	502	594	420	344
1866	576	.....	.....	.....	.....	.....	303	629	222	460	425	94
1867	362	.....	.....	.....	.....	.....	.....	.....	.....	.....	150	350
1868	292	350	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1869	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1870	.....	.....	.....	.....	.....	.....	.....	.....	.....	284	489	553
1871	402	.....	.....	.....	.....	.....	373	507	774	600	245	296
1872	332	460	181	.....	.....	.....	627	234	13	19	.....	136
1873	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1874	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	469	704
1875	894	.....	.....	.....	.....	.....	.....	.....	.....	9	.....	90
1876	334	.....	.....	.....	.....	.....	.....	.....	484	758	.....	440
1877	319	112	.....	.....	.....	.....	.....	.....	258	346	.....	.....
1878	.....	.....	.....	.....	.....	.....	.....	.....	207	465	.....	.....
1879	.....	.....	.....	.....	.....	.....	.....	.....	373	927	794	521
1880	.....	.....	.....	.....	.....	.....	620	877	920	895	795	819
1881	571	.....	.....	.....	.....	.....	.....	.....	.....	672	604	197
1882	.....	.....	.....	.....	.....	.....	.....	.....	516	691	730	675
1883	654	34	.....	.....	.....	.....	587	187	909	808	795	800
1884	.....	.....	.....	.....	.....	.....	.....	.....	541	914	825	42
1885	.....	.....	.....	.....	.....	.....	.....	.....	760	652	.....	.....
1886	.....	.....	.....	.....	.....	.....	.....	334	882	849	326	.....
1887	.....	.....	.....	.....	.....	.....	.....	.....	635	804	631	334
1888	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1889	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1890	.....	.....	.....	.....	.....	.....	.....	.....	225	.....	.....	.....
1891	.....	.....	.....	.....	.....	.....	.....	129	797	783	695	436
1892	.....	.....	.....	.....	.....	.....	.....	23	770	870	301	475

TABLE No. 7.

STEAM-POWER REQUIRED TO MAKE A UNIFORM TOTAL OF 1 000 H. P. 24 HOURS A DAY IN A MILL COMMANDING 500 SQ. MILES OF NEW ENGLAND DRAINAGE AREA, WITH RESERVOIRS TO THE EXTENT OF 10 000 000 000 CU. FT. HEAD, 15 FT. POWER EXPRESSED IN HORSE-POWER, 24 HOURS A DAY.

	Jan.	Feb.	March.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1863												
1864												
1865												
1866							237	629	222	460	425	94
1867	362											
1868												
1869												
1870												
1871								107	774	600	245	296
1872	332	460	181				627	234	13	19		136
1873												
1874												
1875												
1876												
1877												
1878												
1879												115
1880							620	877	920	895	795	819
1881	571											
1882											303	675
1883	654	34					587	187	909	808	795	800
1884												
1885									581	652		
1886												
1887												
1888												
1889												
1890												
1891												
1892								23	770	870	301	475

TABLE No. 8.

COST OF STEAM-POWER TO MAINTAIN 1 000 H. P. 24 HOURS A DAY IN A MILL COMMANDING 500 SQ. MILES OF NEW ENGLAND DRAINAGE AREA, WITH RESERVOIRS OF DIFFERENT SIZES. HEAD, 15 FT.

	No reservoir.	3 000 000 000.	5 000 000 000.	10 000 000 000.
1863	$12 183.60	0.00	.....	.....
1864	28 227.30	$16 060.50	$8 084.40	.....
1865	33 777.00	22 278.60	14 278.80	.....
1866	26 601.00	21 308.40	21 308.40	$16 155.90
1867	27 531.90	14 434.50	6 682.20	2 932.20
1868	19 847.70	6 337.20	4 885.20	.....
1869	17 250.30	4 095.00	0.00	.....
1870	29 709.00	18 132.30	10 196.10	.....
1871	32 108.10	24 814.50	24 814.50	15 641.40
1872	22 907.70	15 470.40	15 470.40	15 470.40
1873	16 873.80	4 781.40	.....	.....
1874	28 468.20	16 760.40	8 868.00	.....
1875	29 640.00	15 545.40	7 745.40	.....
1876	35 713.80	23 547 90	15 592.80	.....
1877	29 372.40	15 777.00	8 094.30	.....
1878	24 633.90	13 039.80	5 319.00	.....
1879	41 750.40	27 998.70	20 198.70	897.00
1880	49 798.50	38 184.30	38 184.30	38 184 30
1881	35 301.90	23 580.30	15 762.00	4 453.80
1882	40 862.70	28 449.00	20 402.20	7 335.00
1883	44 673.90	36 774.00	36 774.00	36 774.00
1884	37 184.40	25 651.50	17 976.00	.....
1885	29 765.40	18 057.60	11 209.20	9 813.00
1886	38 216.70	26 516.70	18 708.90	.....
1887	38 038.80	26 585.70	18 561.90	.....
1888	16 017.60	4 560.30	.....	.....
1889	7 296.60	.....	.....	.....
1890	20 970.90	9 400.20	1 687.50	.....
1891	41 838.60	30 004.50	21 969.90	.....
1892	36 651.30	23 274.30	18 934.80	18 940.80
Totals	$893 213.40	$551 420 40	$391 708.90	$166 597.80
Averages	29 773.78	18 380.68	13 056.96	5 553.26

Subtracting the average of the third, fourth and fifth columns, respectively, from that of the second, we find:

The annual saving consequent upon a reservoir capacity of 3 000 000 000..... $11 393 10

The annual saving consequent upon a reservoir capacity of 5 000 000 000..... 16 716 82

The annual saving consequent upon a reservoir capacity of 10 000 000 000..... 24 220 52

These figures show what we knew perfectly well before, viz., that the benefit of reservoirs to one single establishment, or one mill privilege, is hardly ever sufficient to warrant the expense of their con-

struction. The total head to be served must usually be as much as 100 ft., to bring such projects within the range of feasibility. Exceptions occur when the level of natural lakes can be raised at slight expense, but such opportunities in New England are now nearly all appropriated.

Suppose six such establishments lying on the same stream with an aggregate fall of 90 ft. In this case the aggregate saving would be :

For a reservoir of	3 000 000 000.....	$68 358 60	per annum.
" "	5 000 000 000.....	100 300 92	"
" "	10 000 000 000.....	145 323 12	"

and the cost at which such reservoirs would be a 10% investment would be—

For	3 000 000 000.....	$683 586	or $228	per 1 000 000
"	5 000 000 000.....	1 003 009	" 201	" "
"	10 000 000 000.....	1 453 231	" 145	" "

At the highest of these figures, opportunities for storage can usually be found in most New England river valleys. At the lowest their occurrence cannot usually be counted on.

Let us suppose a series of establishments on the same stream, circumstanced as above with reference to power, having an aggregate fall of 150 ft., which is a case by no means unusual. In this case the cost at which reservoirs would be a 10% investment, would be $380 per 1 000 000 for 3 000 000 000; $334 per 1 000 000 for 5 000 000 000; and $242 per 1 000 000 for 10 000 000 000. Of course, any capacity under 3 000 000 000 would be obtained upon equally or more advantageous terms. It would be rare, indeed, if opportunities for storage could not be obtained at these figures.

These tables throw light upon other controverted questions upon the subject of power. How often do we hear it said that water-power has had its day. It is going out of use. It is being superseded by steam, etc.

The total volume of water discharged by the 500 sq. miles of drainage ground in the assumed period of 30 years would be 769 200 000 000 cu. ft. The quantity used for power in connection with steam is 482 600 000 000. Consider, now, how much could have been used if no steam had been employed. A mill drawing 500 000 000 cu. ft. per month, which

is something like 200 cu. ft. per second, exclusive of wastage, would have been short of water 63 times during the 30-year period, so that 500 000 000 a month is the most that could have been profitably used by such a mill. The entire volume used by such a mill would have been 168 166 000 000 cu. ft. against 482 600 000 000 available when used in connection with steam. We are entitled to say then, that of the total flow from a given drainage ground, or given stream, approximately three times as much can be made available for power by the aid of steam as can be used to any rational purpose without it. So far from water-power being superseded or rivaled by steam, it is only in connection with steam that it attains its most complete development.

Another point may be adverted to in this connection. It is sometimes asserted that it is cheaper to use steam alone than to run steam in connection with water-power.

Compare the cost of power in a mill running as assumed in these tables with that in a mill using the same power obtained from steam alone. We will roughly estimate the cost of water-power plant at $80 per horse-power, say, $80 000 for 1 000 H. P.

We will admit that steam-power in an engine used to make up deficiencies of water-power costs 50% more than in an engine running constantly at full load; that is, 20% more than assumed in Table No. 8.

For the mill using water and steam the account would stand thus:

Cost of steam 20% more than in Table No. 8.....	$35 728 54
Interest on cost of water-power plant at 5%.....	4 000 00
Sinking fund for entire renewal of water-power plant in 30 years at 5%.....	1 130 60
Maintenance and care of water-power.....	2 500 00
Total.....	$43 359 14

For the mill using steam alone:
1 000 H. P., 308 days, 24 hours per day, at 1 cent per hour. $739 20

This sum might reasonably be abated by the advantage which this mill would have in obtaining heat by waste steam during the winter, an advantage not possessed by the other, which has no use for steam-power during the winter. But making due allowance for this advantage, the cost is in favor of the mill using water-power by 50 per cent.

These tables also throw some light on the question of the absolute cash value of water-power, a subject on which wildly diverging opinions

are often expressed. Thus, if we can correctly determine the cost of power for an establishment such as is contemplated, when run wholly by steam, and again when run in connection with the water-power, the difference, as it appears to the writer, may be accepted as the accurate value of the water-power. If, for instance, we could assign $6 000 per annum as the value of the advantage possessed by the first-named establishment in respect to heating, the case would stand thus:

Cost per annum for steam-power alone	$67 920 00
" " steam and water.....	43 359 14
Value of water per annum.....	$24 560 86

This sum is what the mill could afford to pay annually for the water used for power, and still stand upon an equal footing with an establishment run wholly by steam at the same place.

The foregoing remarks do not apply to the case of a mill engaged in bleaching, dyeing and similar operations requiring great amounts of heat.

Pondage.—Hardly less important than storage, or the retention of flood water for use during times of scarcity, is pondage or the retention of the flow of a stream during non-working hours, for use during working hours.

To have supposed the water used during the working hours of the secular day, in the preceding computations, would have introduced too large an element of hypothesis and made the matter too complex. This embarrassment we have avoided by assuming the mill to run throughout the 24 hours. Flour mills, lumber mills and paper mills usually run in this manner, and require no pondage. Textile mills more commonly run in the daylight and during the working hours established by law or custom.

We have taken the maximum limit of the use of water at 800 cu. ft. per second, which is very nearly the average flow of the stream for the entire period considered. This is as large a use as is ordinarily made by an establishment running 24 hours a day, though, no doubt, a still larger use might in some cases be made with advantage. An establishment running 10 hours a day, and designed to make full use of this quantity of water, must be organized upon an entirely different basis. All its buildings, machinery, boilers, engines, water-wheels, raceways, etc., would require to be on a scale of magnitude about 2½ times as great, and it would require pondage to hold 800

cu. ft. per second during the time that the wheels are not running, viz., about 46 800 seconds each day ; that is to say, about 37 500 000 cu. ft. of pondage. The maximum limit of the use of water would be $800 \times 24 \div 10 = 1\ 920$ cu. ft. per second.

In the actual running of mills it is very seldom practicable to use the water to this extent. The amount of pondage pertaining to a mill privilege depends usually upon the height of its dam and the declivity of the bed of the stream above the dam, and has no relation to the requirements for power. Practically, the mill makes use of what pondage it has, and, beyond this, allows the night flow to run to waste. The flow of Sundays and holidays would run to waste even with the pondage of 37 500 000, as given above. To avoid this waste, we should ordinarily have to hold the flow for about 40 hours, requiring $800 \times 40 \times 3\ 600 = 115\ 200\ 000$ cu. ft. of pondage.

To exhibit the value of pondage by the same rigorously exact methods as have been adopted with reference to storage, Tables Nos. 9 and 10 have been computed. These tables assume an establishment with the same drainage area and the same fall as in the former case, running 10 hours a day and stopping one hour at noon, its maximum use of water being 1 000 cu. ft. per second. In Table No. 9 the first column is the natural flow of the stream in cubic feet per second, for each 25 ft. from 50 to 1 000. The second column gives the quantity of water which the mill is able to use with any given flow of the stream and 10 000 000 cu. ft. of pondage. Thus, with 50 cu. ft. per second, the entire flow of 168 hours can be saved and drawn in the 60 working hours, giving a draft of 140 cu. ft. per second. The same with a flow of 75 cu. ft. per second. With a flow of 100 cu. ft. per second there is a little wastage of the Sunday flow. Above 214 cu. ft. per second the Sunday flow is wholly wasted. Above this stage the computation is as follows, say for a flow of 225 cu. ft. per second :

In pond at starting of mill.....			10 000 000 cu. ft.
One hour's flow held at noon = 225 × 3 600...			810 000 "
Pondage drawn in 10 hours or 36 000 seconds.....	= 300 cu. ft.	per second	10 810 000 "
Add natural flow.....	225 "	"	
Total draft of mill.	525 "	"	

The third column gives the gain from pondage in cubic feet per second ; the fourth, the same in horse-power ; the fifth, the value of

equivalent steam-power per diem, taking steam-power, as in the former case, at 12½ cents per horse-power per day. When the draft reaches 1 000 cu. ft. per second, the gain from pondage begins to diminish; and when the natural flow of the stream reaches 1 000 cu. ft. per second, there is no further gain from pondage. The sixth, seventh, eighth and ninth columns are the same as the second, third, fourth and fifth, except that the former refer to a pondage of 20 000 000, the latter to one of 10 000 000.

TABLE No. 9.

VALUE OF PONDAGE TO A MILL COMMANDING 500 SQ. MILES OF NEW ENGLAND DRAINAGE AREA, RUNNING 10 HOURS A DAY AND CAPABLE OF USING 1 000 CU. FT. OF WATER PER SECOND. HEAD, 15 FT.

Flow of stream in cubic feet per second.	10 000 000 cu. ft. of pondage.				20 000 000 cu. ft. of pondage.			
	Draft of mill in cubic feet per second.	Gain from pondage.		Cost per diem of steam-power equivalent to gain from pondage.	Draft of mill in cubic feet per second.	Gain from pondage.		Cost per diem of steam-power equivalent to gain from pondage.
		Cubic feet per second.	Horse-power.			Cubic feet per second.	Horse-power.	
50	140	90	112	$14.00	140	90	112	$14.00
75	210	135	169	21.12	210	135	169	21 12
100	265	165	206	25.75	280	180	225	28.12
125	319	194	242	30.25	350	225	281	35.12
150	372	222	277	34.62	420	270	337	42.12
175	428	253	316	39.50	475	300	375	46.87
200	483	283	354	44.25	530	330	412	51.50
225	525	300	375	46.87	584	359	449	56.12
250	553	303	379	47.37	639	389	486	60.75
275	580	305	380	47.50	693	418	522	65.25
300	608	308	385	48.12	748	448	560	70.00
325	635	310	387	48.37	802	477	595	74.50
350	663	313	391	48.87	857	507	634	79.25
375	690	315	394	49.25	911	536	670	83.75
400	718	318	397	49.62	960	560	700	87.50
425	745	320	400	50.00	995	570	712	89.00
450	773	323	404	50.50	1 000	550	687	85.87
475	800	325	406	50.75	1 000	525	656	82.00
500	828	328	410	51.25	1 000	500	625	78.12
525	856	331	414	51.75	1 000	475	594	74.25
550	883	333	416	52.00	1 000	450	562	70.25
575	911	336	420	52.50	1 000	425	531	66.37
600	938	338	422	52.75	1 000	400	500	62.50
625	965	340	425	53.12	1 000	375	469	58.62
650	993	343	429	53.62	1 000	350	437	54.62
675	1 000	325	406	50.75	1 000	325	406	50.75
700	1 000	300	375	46.87	1 000	300	375	46.87
725	1 000	275	344	43.00	1 000	275	344	43.00
750	1 000	250	312	39.00	1 000	250	312	39.00
775	1 000	225	281	35.12	1 000	225	281	35.12
800	1 000	200	250	31.25	1 000	200	250	31.25
825	1 000	175	219	27.37	1 000	175	219	27.37
850	1 000	150	187	23.37	1 000	150	187	23.37
875	1 000	125	156	19.50	1 000	125	156	19.50
900	1 000	100	125	15.62	1 000	100	125	15.62
925	1 000	75	94	11.75	1 000	75	94	11.75
950	1 000	50	62	7.75	1 000	50	62	7.75
975	1 000	25	31	3.87	1 000	25	31	3.87
1 000	1 000	0	0	0	1 000	0	0	0

Table No. 10 gives the aggregate value of pondage for each year of the series. It is computed from Table No. 9 and Table No. 1, with the aid of the calender of the several years. Thus, for the year 1865, with a pondage of 10 000 000:

January.—Average flow, 932. Value of pondage per diem, $10 63.
Number of working days, 26.....26 x $10 63 = $276 38
February. Flow, 835. Days, 23. Value pondage, $25 77...23 x 25 77 = 592 71
June. " 149. " 26. " " 34 45...26 x 34 45 = 895 70
July. " 199. " 25. " " 44 06...25 x 44 06 = 1 101 50
August. " 204. " 27. " " 44 67...27 x 44 67 = 1 206 09
September. " 201. " 26. " " 44 35...26 x 44 35 = 1 153 10
October. " 303. " 26. " " 48 15...26 x 48 15 = 1 251 90
November. " 448. " 25. " " 50 46...25 x 50 46 = 1 261 50
December. " 490. " 25. " " 51 05...25 x 51 05 = 1 276 25

Value of pondage, 10 000 000 cu. ft. for 1865.....$9 015 13

TABLE No. 10.

VALUE OF PONDAGE FOR A SERIES OF YEARS TO A MILL COMMANDING 500 SQ. MILES OF NEW ENGLAND DRAINAGE AREA, AND RUNNING 10 HOURS A DAY. HEAD, 15 FT.

Year.	SAVING IN STEAM POWER WITH—		Year.	SAVING IN STEAM POWER WITH—	
	10 000 000 cu. ft. of pondage.	20 000 000 cu. ft. of pondage.		10 000 000 cu. ft. of pondage.	20 000 000 cu. ft. of pondage.
1863.....	$7 382.39	$9 806.91	1878.....	$5 318.63	$8 329.41
1864.....	10 989.73	12 965.00	1879.....	8 421.89	11 587.40
1865.....	9 015.13	12 008.18	1880.....	7 269.06	8 438.36
1866.....	13 778.64	19 123.17	1881.....	7 328.73	8 878.23
1867.....	9 037.74	13 409.27	1882.....	6 801.70	10 117.82
1868.....	10 013.91	13 788.23	1883.....	9 577.74	11 050.22
1869.....	7 967.86	10 799.78	1884.....	7 820.73	9 058.85
1870.....	9 807.46	14 589.50	1885.....	5 481.03	6 837.64
1871.....	11 355.77	13 882.46	1886.....	6 865.17	8 286.00
1872.....	12 204.44	15 942.47	1887.....	7 585.85	10 127.32
1873.....	6 982.52	8 595.21	1888.....	4 362.39	5 643.21
1874.....	8 744.71	11 863.72	1889.....	5 769.89	8 091.06
1875.....	8 381.44	11 748 32	1890.....	5 031.00	6 888.21
1876.....	8 607.26	11 576 60	1891.....	8 501.88	12 117.79
1877.....	6 922.36	9 498.21	1892.....	9 879.13	12 500.84
Average.....				$8 240.31	$10 918.31

The recently established holidays, Labor Day and Decoration Day, are not counted. Table No. 10 being given, not for reference, but for illustration, the computations have not been checked. They are substantially correct, but not free from trifling errors. The average result for the entire period of 30 years is $8 240 31 for a pondage of 10 000 000,

and $10 918 31 for a pondage of 20 000 000. These sums represent the average annual benefit of the respective amounts of pondage to an establishment running under the conditions supposed, as compared with one running under the same conditions without pondage. These results would appear larger if we should assume a larger maximum use of water; that is, if we assume the establishment capable of using 1 200 or 1 500 cu. ft. of water per second, instead of 1 000.

In the case of a series of mills running 10 hours a day, and supplied by storage reservoirs, it often happens that the daily discharge from the reservoirs does not reach the mills within the working hours, and more or less pondage is necessary in order to make the discharge of the reservoirs available and avoid waste. No greater amount of pondage, however, is necessary for this purpose than would be required to make an equally complete use of the natural flow of the same stream; generally less.

In the case of a series of mills occupying different falls on the same stream, the pond of each mill reaching to the dam of the next above, it is sometimes advantageous to provide a large pond at the head of the line. Each mill, then, on starting, draws from its own pondage only till it receives water from the next above, and the single pond serves the whole series. This is the case at Gardiner, Me., where there is a succession of dams supplied by the Cobossee Contee River. This stream, draining about 215 sq. miles, has some 2 500 000 000 or 3 000 000 000 cu. ft. of storage capacity, and has a pond at the head of the series of dams capable of holding the discharge of the reservoirs while the mills are stopped. This is usually only on Sundays and holidays, the mills being mostly paper mills.

The most fortunate condition is when the same basin serves both for storage and pondage. This case occurs at Rockville, in Connecticut. Here, Lake Shenipsit, near the head of the Hockanum River, has a drainage area of some 15 sq. miles, and is controlled by a dam to the extent of some 24 ft., holding practically the entire flow of the drainage ground. Below the dam is a series of mill privileges, with an aggregate fall of more than 250 ft. The declivity is so great that the pondage appurtenant to each dam is very small. These are mostly textile mills, and run but 10 hours a day. The reservoir gates are opened only during working hours. As soon as any mill commences to draw from its pond the mill above commences to discharge into the

same, and very little water runs to waste. Very rarely does so small an extent of drainage ground furnish power for so large an amount of machinery.

The construction of a system of storage reservoirs, or any similar enterprise, in the common interest of a number of mill owners, presupposes a concert of action and a spirit of mutual concession and accommodation among the parties very rarely met with. Mutual jealousies and bickerings over trivial points usually prove fatal obstacles to such projects. For this reason it is customary and advisable for the parties in interest before commencing such an undertaking to appoint a board of arbitration empowered to apportion the expense and to adjust all questions requiring mutual agreement of the parties.

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689.

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RECLAMATION OF THE POTOMAC FLATS AT WASHINGTON, D. C.*

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READ DECEMBER 20TH, 1893.

The Potomac River from Little Falls to its mouth is a tidal stream, the tidal range being 3 ft. at Washington, and about $1\frac{1}{2}$ ft. at its mouth. Above the Falls the tides have no effect whatever. The lower part of the river is broad, but there is a good channel all the way to Washington, rather shoal in a few places, and bordered on the sides with large areas of flats or marshes which support a rank growth of eel grass.

Ever since we have had definite knowledge of the river, such as is derivable from maps, etc., there have been no less than two channels in the harbor of Washington, and a part of the time three. The most southerly one of these, called the Virginia Channel, and naturally the widest and deepest, followed along the Virginia shore, and passed between it and Mason's Island (now called Analostan Island). When the upper end of this channel was closed by the construction of a dam or causeway, between the island and the Virginia shore, in 1806, it was

* Discussions on this paper received before May 1st, 1894, will be published in a subsequent number.

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forced over to the Georgetown side and became sometimes known as the Georgetown Channel. The causeway was built with a view to deepening the channel at the cross-over below Analostan Island. It was already deep along the Georgetown front. The island became, in fact, a training dike, but it was not long enough. The shoal place remained shoal, as might have been anticipated. At the same time a large area of flats was created in the dead water below the island.

A second channel, sometimes called the "swash," branched off from the Washington Channel, opposite the foot of Sixth Street, pursued a southwest course, passed through the middle of what is now the causeway of Long Bridge, and united with the Virginia Channel a short distance below Easby's Point. In the early days this seems to have been a fairly good channel, having a navigable depth of 6 ft. at mean low tide. This channel is shown by the maps to have filled up, to a great extent, by 1834, the date of the Kearney survey, and the subsequent construction of Long Bridge causeway caused its total obliteration. There is a popular notion that the construction of this causeway produced the filling up of this channel. This is an error. The causeway was not built until 1836, and at that date it had already filled up. That it accelerated the process afterward is doubtless true; but it is a perversion of the terms, cause and effect, to attribute the original filling-up process to the causeway.

The third channel, called the Washington Channel, branched off from the main channel opposite the mouth of the Eastern Branch, and followed along the Washington shore to Easby's Point where it again united with the Virginia Channel. There is a popular notion that this channel was deep at its upper end, so that vessels of good size could follow it all the way to Georgetown. But it must have been a very long time ago when such was the case. The survey of 1792 (Plate III) shows that there was only a navigable depth of 3 ft. at mean tide, which corresponds to $1\frac{1}{2}$ ft. at mean low tide. General Braddock is said to have landed his army in the cove below Easby's Point, but even this might be true and the depth of water not great. Surveys made subsequent to 1792 shows that the little channel continued to fill up, so that by 1834 it had entirely filled up—the remains of it existed in the shape of a narrow drain between the shore and the flats outside of it. By 1862 the upper part of the channel had disappeared entirely and was lost in a broad expanse of flats. There only remained the two channels, the

Virginia, or Georgetown, Channel on the south side, and the Washington Channel, which practically terminated at Long Bridge.

The same causes that produced the shoaling of the channels produced the formation known as the Potomac Flats. They are the direct result of the physical conditions at the time existing. At Easby's Point, which marks their upper limit, the river is only about 900 ft. wide, estimating to Analostan Island, which was united to the Virginia shore by the dam already referred to, while immediately below it was from 5 000 to 6 000 ft. This sudden expansion in width naturally caused a deposit of silt in this part of the river in times of freshets, filling the channel to some extent, but filling still more in the eddies and dead water behind Easby's Point. The drainage area of the Potomac is about 20 000 sq. miles ; consequently, freshets attain huge proportions. The river rises at such times to a height of 40 to 45 ft. at the Falls and becomes a huge torrent, the discharge having been roughly estimated at about 380 000 cu. ft. per second. An immense amount of silt is brought down with it, which finds a resting place during the progress of the freshet in the eddies behind projecting points, and, as the freshet subsides, quantities are deposited in the channels themselves.

In this way the channels have been filling up, requiring frequent dredging to maintain the requisite depth for commerce, and the Potomac flats have been growing year by year until they had become a nuisance to the city. Of late years this nuisance had been aggravated by the large amount of sewage that was discharged on them. The city had been growing, and the question of sewage disposal had not engaged the attention of the community or Congress to the extent that it should. A grave error in sewage disposal was committed when the old canal that ran along B Street was converted into a sewer, with its discharge on the flats at the foot of Seventeenth Street. The greater part of the flats being covered at about half tide, the sewage spread out on them, and, when the tide went out, left it in putrid masses, exposed to the direct rays of the sun. The vicinity of the outlet of this sewer became a pestilential swamp, the stench from which was at times almost intolerable. The Presidential mansion, being distant only about half a mile, got the full benefit of the condition of affairs when the wind was from the south.

To correct this growing evil was a problem that had engaged the

attention of the public for some time, and various plans had been suggested. A change in the disposal of the sewage would only ameliorate the conditions. It would not cure them. The flats would still be there, and the nuisance would be perpetuated, though in a less objectionable form. The question was: Should the flats be reclaimed by filling on top of them so as to raise the surface above the level of overflow, or should they be dug out, thereby creating a large area of deep water in front of the city, or should they be diked in and pumped out? All these plans had their advocates, and all were more or less discussed in the public press. To undertake to discuss the merits and demerits of the various plans would not be practicable within the limits of this paper. It will be sufficient to note the salient features of those that were most seriously considered, and a little more in detail the one that was adopted.

Mr. Alfred L. Rives, a civil engineer, was one of the first to treat the subject from a professional standpoint. In his report, transmitted to Congress by the Secretary of the Interior in 1857, he submitted plans and estimates for an iron suspension and stone arch bridge across the Potomac, on the site of the existing Long Bridge. He states that the plans for a permanent bridge at Washington are so intimately connected with the project for the improvement of the navigation of the river that one cannot be properly matured without due consideration of the other. He might have added that the improvement of navigation was also in the same way intimately connected with the reclamation of the flats. His plan of improvement was to open the old swash channel, fill out to the edge of it on the Washington side, turn the waters of the Virginia Channel into the new one by means of a breakwater built out from the lower end of Analostan Island, and reconstruct Long Bridge. This plan would reclaim 166 acres on the Washington side, but would have brought about just the conditions for forming a much larger area for flats on the opposite side of the river. A peculiar feature was that very little dredging was to be done in the formation of the new channel; most of it was to be formed by the supposed scour that would take place. The flats were to be filled by material taken from the grading of streets, etc.

Another plan, wholly dissimilar from that of Mr. Rives, was proposed by the Board of Survey of 1872. This was to advance the city front along the whole line out to the Virginia Channel; to construct

a bulkhead along the entire front, with docks in number and size sufficient to accommodate commerce, and to fill in the area between the new and the old fronts, chiefly with dredgings taken from the river, which was to be deepened to 23 ft. at mean low tide. This plan would reclaim 1 050 acres, all of which was to be laid out in streets and lots, by extending the existing streets over the area to be reclaimed. There were many other minor details connected with this plan, which it is unnecessary to refer to. It was a work of great magnitude as planned.

In 1879, Major Twining, at the time Engineer Commissioner of the District of Columbia, submitted a plan for improving navigation and filling the flats, which was a modification of the plan of the Board of Survey. His plan advanced the city front, but not as far. The Washington Channel was to be kept open as far as Long Bridge, which was a practical extension of the Board's winter harbor. He introduced an entirely new feature which had not up to that time been considered, viz., the flushing ponds or lakes. Under his plan the Washington Channel became a tidal arm of the river, closed at its upper end from all connection with the river. To overcome the objections urged, that this comparatively small body of water would become stagnant and polluted, he proposed to occupy about 114 acres of the space between Long Bridge and the sewer canal with lakes, which were to be arranged with inlet and outlet gates, working automatically with the tides, taking the water in from the Virginia Channel on the flood, and discharging it into the head of the Washington Channel on the ebb. The entire space between the bridge and sewer canal was to be converted into a park, the lakes forming an ornamental feature; the other spaces, below the bridge, and above the sewer canal, were to be laid out in streets and lots, the latter were to be sold. He proposed to fill the entire area to a height of 4 ft. above the freshet line of 1877. The plan reclaimed about 528 acres.

In January, 1882, Mr. S. T. Abert, civil engineer, submitted a plan, which was the same in its general features as that of Major Twining, except that he proposed to fill the flats only to the level of 6 ft. above low tide, and protect it from overflow by a surrounding embankment.

In February, 1882, a board of engineers, consisting of Lieutenant-Colonels Q. A. Gillmore, William P. Craighill, and C. B. Comstock, Corps of Engineers, U. S. A., submitted a plan combining sub-

stantially and in its essential features the harbor and shore lines of Mr. Abert's plan, with the low-grade filling and surrounding embankment of that plan, below Long Bridge, with Twining's high-grade filling and sluicing ponds between Long Bridge and Easby's Point. The Board regarded the rebuilding of Long Bridge with wider spans on piers, offering less obstruction to the flow of water, as essential.

The plan of the Board of Engineers was the one adopted by Congress, and has been nearly carried out. The low-grade filling with the surrounding embankment below Long Bridge was subsequently modified to a fill 3 ft above the freshet line of 1877, along the axis of this area, which was tongue-shaped in plan, sloping off to about 6 ft. above low tide at the margin on either side. For the flushing ponds, of which there were four, all communicating with each other, as proposed by Twining, a single lake or reservoir was substituted. The total area of flats reclaimed is about 625 acres exclusive of the reservoir, requiring about 12 329 000 cu. yds. of material.

The act making the appropriation provided for improving the navigation, the raising of the flats, and the establishment of the harbor lines. The law itself, therefore, contemplated that the material, or at least most of it, should be taken from the bed of the river; but whether contemplated or not, it was the only suitable place from which it could be obtained. A small amount might be had from Observatory Hill, but the leveling of this hill, except to a small extent, was neither desirable nor advantageous. It was necessary to procure it from the bed of the river. Besides being the most economical method of obtaining it, it accomplished two objects at the same time—the improvement of navigation, and the reclamation of the flats.

The first contract was for dredging a channel 20 ft. deep and about 400 ft. wide, about two-thirds of the spoils to be deposited on the flats between the sewer canal and Easby's Point, and the remainder on that part of them below Long Bridge to a grade of about 6 ft. above mean low tide, the object being to reclaim as much of the flats from overflow as practicable with the funds available.

The plan at first adopted was to dredge the material in the ordinary way by means of dipper and clam-shell dredges, convey it in bottom-dumping scows to a receiving basin and there discharge it. At the receiving basin it was again taken up by a clam-shell dredge and deposited into cars. The latter were hauled to the dumping ground by a locomotive.

This method of doing the work not only necessitated the handling of the material twice, but required the construction of long lines of trestle-work, all of which became useless as soon as the work was finished. The trestle on which the railroad ran was built on piles, most of them about 60 to 70 ft. long, cut off, after being driven, at about 10 ft. above the surface of the flats, and capped with 12 x 12-in. timbers. On these caps longitudinal timbers of the same dimensions were laid, and on the latter the rails. The cars were double side-dumpers and had a capacity of 10 cu. yds. each. They were hauled out in trains of 10 cars each, and the contents of a whole train were dumped at one time. For putting the material on the flats above Long Bridge there were constructed about 13 000 lin. ft. of trestle, including the double-track part at the receiving basin. There were deposited on this section of the flats about 862 872 cu. yds., scow measurement, which gives only about 66 cu. yds. per linear foot of track built. The tracks at the receiving basin were set at about 4 ft. above mean low tide, to facilitate the loading of the cars. From this point the grade of the tracks rose until the level of 12 ft. above mean low tide was reached. Several branch tracks started from near the basin at a low level. This was a faulty construction and required more track than if they had started from a higher level.

Below Long Bridge the track was a single line, and most of the material was dumped from a height of about 12 ft. above the level of the flats; 557 000 cu. yds. were dumped from a length of 7 400 ft. of trestle, which gave about 75 cu. yds. per linear foot. In all, there were built over 20 000 lin. ft. of railroad trestle, and there were deposited on the flats by this process 1 418 771 cu. yds., which, when spread out, covered about 120 acres to a depth of 6 ft. above mean low tide. The trestle, exclusive of rails, must have cost $30 000, or at the rate of about two cents per cubic yard of material handled. After the material was dumped from the cars, it was found necessary to level it down by means of a force pump. A Worthington pump was used for this purpose, the water being forced through a 4-in. wrought-iron pipe, at the end of which a small piece of hose, with nozzle, was attached. This method of leveling the material was not altogether satisfactory. It would level a sharp ridge well enough, but if a large quantity of material had to be moved any great distance, the pump was not effective.

The material was dredged from the river at an average depth, say, of 18 ft. below the surface of mean tide, and raised by the first operation of dredging to a height of about 8 ft. above that level, say, 26 ft. in all. It was then towed to the basin and dropped into it. Again it was picked up and loaded into cars, being raised again about 28 ft. for this purpose. It was then hauled out to the dump, being raised about 6 ft. more in taking it to the dumping ground. The material was thus lifted vertically about 60 ft., whereas the average difference in level between the material in the river and on the flats was only about 21 ft. It will thus be seen that an enormous waste of power takes place in this method of handling such material.

The price paid for material dredged from the river-bed and deposited on the flats in the manner just described was 21.2 cents per cubic yard (scow measurement). It will be seen farther on that this was largely in excess of the prices paid subsequently for the same work done by a different method. The price, 21.2 cents, includes all the profits to the contractor.

Simultaneously with the dredging of the channel and filling the flats, the construction of embankments was begun along the margin. This work it was decided to begin at an early day, as it would afford some protection to the material deposited on the flats in times of freshets, and the early construction of these embankments would enable a better and more economical method to be pursued in filling behind them.

The embankments had to be formed for the most part in water that varied from 2 to 4 ft. deep at mean low tide, corresponding to 5 to 7 ft. at ordinary high tides. In some places the depth was less, and in others more. The material along the lines of the embankments varied. Most of it was a soft alluvial, easily handled, and generally good for the purpose. Some of it was very soft and compressible. Only a small percentage of it consisted largely of sand, and this was not difficult to handle.

The embankments were built in the following manner:

An ordinary light-draught clam-shell dredge would make a cut 6 ft. deep at mean low tide, and wide enough to enable her to work conveniently, depositing the spoils on the side toward the proposed fill.

A low embankment would be thus formed with a berm of about 2 or 3 ft. between the foot of the outer slope and the edge of the cut. In

some places where the water was deep, or the material good, a second cut would be made, widening the trench and depositing the material over and beyond the ridge first formed. Stone was then thrown into the trench, forming a ridge about 9 ft. high with slopes of 1 on 1, as shown on the accompanying sketch (Plate VI). The trench was then further widened and the outer part deepened, using a dredge with a longer boom, or a dredge in connection with a chute carried on a scow which floated alongside and moved with it.

An endless chain dredge was used on a part of the work after the first cut had been made. It gave very satisfactory results. This machine was rigged with an endless chain excavator at the bow, by which the material was lifted to a height of about 12 ft. and then fell into a hopper, from which it was fed into a "conveyor," by means of which it was carried to the place of deposit over and beyond the embankment first formed. The "conveyor" was a line of little cars that formed the links of an endless chain, supported on a frame built on scows moored alongside the dredge, and operated from the latter.

Where there was not enough material in the trench, it was brought there from the channel or some other part of the work in scows. But comparatively a small quantity had to be secured in this way, as on that part of the flats, where the embankments were to be the highest, the water was generally shoalest. No difficulty was experienced in raising the embankments to a height of 12 to 15 ft. above low tide, which was from 14 to 18 ft. above the level of the original bottom.

The slopes which the material took varied with the character of soil and the kind of dredge used. The ordinary clam-shell dredge working without a chute, gave the steepest slopes.

Worked in connection with a chute, the material, softened by handling and aided by a stream of water, acquired such velocity in sliding down the chute that it would sometimes run 200 or 300 ft. beyond the end of it, giving the embankment on that side a very flat slope. The endless chain dredge also gave very flat slopes. The work of forming these embankments was one requiring considerable time. It could not be done rapidly. Time was needed to allow the material taken from the bottom of the river to dry out and become consolidated.

As the work extended over a period of seven or eight years, there was no lack of time. In two or three bad places a slight forward movement took place, but it was not sufficient to cause any serious trouble,

and was due to the building up of the embankment faster than it should have been at those places. The embankments thus formed had an aggregate length of about 8½ miles, and over this long line the material, of which built, varied considerably. Where there was some sand mixed with alluvium, it would dry out and become firm in a short time; in other places where the bed of the river was soft mud, it took a long time to harden. In one place it was only with the greatest difficulty, by going over it frequently and by allowing plenty of time for it to dry out, that the embankment could be gotten to stand at all. The water in this spot was about 8 ft. deep originally, and the bottom extremely soft. Of course, in that case material for the embankment could not be taken from a trench in front of it, as the water was already too deep. The stone was placed in the usual way and material was brought from some other part of the work, dumped in front of the stone pile, and then redredged and deposited over and beyond it.

As before noted, nearly all the embankments were built by depositing a ridge of stone in the trench. This formed a footing for the embankment and a foundation for the dry wall that was afterward constructed, and was a necessity for the most part to protect the embankment from the abrasive action of the waves. The top of the stone pile, being a little above the level of high tide, protected the bank and prevented it from being washed down.

The two banks of the sewer canal, at the foot of Seventeenth Street, were built without stone footings; but in the canal the water was quiet and the wash of the waves scarcely felt.

The construction of a revetment, or sea wall, as it is sometimes called, was not regularly begun until about all the material needed in the embankments had been laid up. This wall rests on the stone pile at the foot of the embankment, and, starting from low-water level, is 6 ft. high, 5 ft. wide at the base, and 3 ft. wide at the top. The face has a batter of 3 vertical to 1 base. The back of the wall is vertical. This wall is laid up with dry stone, quarried on the banks of the Potomac, about four miles from the work. The stone forming the footing of the embankment and foundation of the wall cost from 69 to 95 cents per cubic yard, or an average of about 87½ cents, delivered in place; that used in the wall proper cost $1 70 per cubic yard delivered on the bank, and $1 45 per cubic yard for labor of laying it up—a total of $3 15 per cubic yard, or about $2 80 per linear foot of wall, making a durable and cheap revetment (see drawings).

The formation of the embankment along the margin of the areas to be filled enabled the hydraulic method of dredging to be used, which was a far cheaper and more satisfactory method of doing the work. The first practical application of the pump for transporting the spoils of dredgers to the place of deposit, known to the writer, was about 1867 or a little earlier, in the construction of the Grand Canal at Amsterdam. The dredging itself was done in the ordinary way, by endless-chain dredgers, the material being deposited into a hopper from which it was fed into a pump attached to the side of the dredge and forced ashore through pipes. The pump was centrifugal, 3 ft. 6 ins. in diameter, laid flat with the top on the level of the water and provided with two inlets protected by valves, one in the bottom for the admission of the water, the other on top for the admission of the material to be transported. The pipes were made of wood, with buoying pieces to float them, and connected by leather joints. The diameter of the pipes was 15 ins.

Four dredges working on this principle were employed in 1867 in excavating a part of the Grand Canal at Amsterdam. The largest one is reported to have been capable of transporting about 1 330 cu. yds. of material, consisting of a mixture of clay and sand, in 12 hours. The distance was about 900 ft., and the lift about 5 ft.

General Gillmore, of the Corps of Engineers, U. S. A., in 1872 operated a suction dredge at the mouth of the St. John's River. At that time it was a novel device as applied to dredging. He used a No. 9 Andrew's centrifugal pump, mounted on a steamer, with two 6-in. suction pipes that reached to the bottom of the river. The ends of the suction pipes were armed with knives or teeth, by means of which the material was stirred up and aided in entering the pipes. The material was sand and was pumped with the water into bins, which were built on the boat for that purpose; here the sand settled, and the water ran off. After filling the bins with sand, the steamer was taken out to sea and the sand discharged. General Gillmore states that he succeeded in raising by means of his pump as much as 45% of solid material, but that 30 to 35% was a fair proportion. The height to which the material was raised was but a few feet above the level of the water. Average time worked per day was six hours. Average quantity of sand excavated per hour, 78 cu. yds. Total quantity in 58 working days, 23 868 cu. yds. Average cost, $31\frac{1}{18}$ cents.

The hydraulic dredges used on the Potomac combined the two operations of excavating the material from the bed of the river and discharging it ashore at the desired place in a single operation.

In principle the machine was essentially a combination of the two processes practiced by the Hollanders in 1867 and by General Gillmore in 1873.

The machine consisted of a large rotary pump mounted on a scow-built vessel, 100 ft. long by 50 ft. beam, held in position by means of spuds in the usual way. From the pump, which is located in the forward part of the vessel, a cast-iron pipe leads to the bow and thence down to the bottom of the river. The part leading down into the water is held in a crane which moves about a center from one side to the other over the bow of the vessel, which is rounded to accommodate this movement, and allows the pipe to describe the arc of a semi-circle 60 ft. in diameter. The pipe is arranged with telescopic joints by means of which the length is regulated so that the lower end is kept at the proper distance from the bottom.

Connected with the pump is a discharge pipe which passes out over the side of the vessel and from thence is carried on pontoons to the shore. Each pontoon carried one section of pipe, set on it longitudinally. The joints of the pipe were made flexible by sections of heavy rubber hose, or by ball joints of cast iron. As may be readily imagined, the rubber joints were a source of considerable expense, as they wore out rapidly and were apt to burst under pressure.

The pump was 8 ft. in diameter, of the Von Schmidt pattern; the suction pipe 22 ins., the discharge pipe 20 ins. in diameter. It was operated by two engines, 22 x 24 ins., which were geared directly to the shaft of the pump. Two more engines, 12 x 18 ins., were used, one to run the turn-table carrying the crane, the other the plows or cutters at the bottom of the suction pipe. Two boilers supplied steam. They were 24 ft. long by 6 ft. diameter, and were called 400 H. P. The usual pressure of steam was 90 lbs.

At the bottom of the suction pipe was a hood or bonnet 8 ft. in diameter, shaped very much like an inverted saucer.

A shaft passing through the center of the hood was provided with radial branches at the bottom. These branches, as well as an iron ring connecting their extremities, were armed with knives or plows, which, by the rotation of the shaft, stirred up the material in the bed of the

river, so that the water moving toward the suction pipe carried the solid matter along with it.

The proportion of solid matter to water pumped through the pipes varied greatly with circumstances. The character of the material, the height and distance of the place of deposit, the velocity at which the pump was run, the dexterity of the man who controlled the action of the cutter, and the distance of the hood from the bottom, all tended to increase or diminish this proportion. Under favorable circumstances 30 to 40%, or even more, of solid matter could be carried, but such large quantities did not give good results as a rule. Too much work was thrown on the pumps, rendering them liable to be overpowered. Ten to 20% was easily handled under favorable circumstances, with the engines making 125 to 130 revolutions per minute. The material that had to be dredged varied greatly, as before stated. In some cases it was soft mud; in others, sand or gravel; sometimes mixtures of these, and sometimes the various kinds would be found in layers. Working in soft material, one of these dredges has averaged about 350 cu. yds. per hour for days, the length of the discharge pipe being over 3 000 ft., and the height above the water-level from 6 to 12 ft.

On one occasion one of these machines put ashore material that had been dredged and deposited in a basin near the edge of the fill, averaging as much as 800 cu. yds. per hour. The material was, in this case, alluvium, brought down the river in freshets, mixed with a little sand, the height of discharge being about 10 ft. above mean low tide, and the distance from the dredge to the place of deposit about 1 200 ft. The material had all been well churned up by being first dredged and then dumped into the basin.

These machines were especially adapted to the work of reclaiming the Potomac Flats. The material, when discharged from the pipes, would spread out in low, conical heaps, the lightest particles being carried off to a considerable distance, while the more heavy stuff found a resting place near the exit from the pipe.

Clay would be discharged in the form of balls from 1 to 5 ins. in diameter, this shape being given to the lumps in their passage through the pipes. Boulders as large as a man's head have been forced through them, but such machines are not adapted to handling gravel and boulders. The latter could be forced through the pipes, but not

economically. Mixed with mud or soft clay, a certain proportion of sand, or even gravel, could be handled with facility, but the amount of the latter must not be great. Pure sand could not be handled with advantage. It acted as a blast and cut away the shell of the pumps in a short time.

A leak would thus be started, which it was sometimes very difficult to find. I have seen 1-in. bolts cut almost through in one day by the sand blast inside the pump.

Working in sand, the discharge pipes were apt to fill up so that their capacity was sometimes reduced 60 to 75 per cent. The sand accumulates first on the bottom of the pipe, where it becomes packed and hard, filling up more and more, until the pump can no longer force water through it. The usual method, when it is found that the pipes are filling up with sand, is to pump pure water through them until they are cleared. But they cannot always be cleared in this way. The sand has been known to pack so hard that water forced through them will not clear them. On one occasion this pumping of water was continued two entire days in an effort to clear the pipes, and it failed. The latter had to be taken apart and the sand scraped out. Water flowing through the discharge pipe of one of these machines has a velocity estimated at from 10 to 15 ft. per second, and yet it could not put the sand in motion.

This method of filling low areas saves all expense and trouble in spreading the material out to the required grade, and it packs much better than when discharged from cars or other vehicles for transportation. When deposited from cars, the mud, if originally soft, becomes softer. A thin coat or shell of dry material forms on top, while that below remains in a jelly-like mass for a long time.

The following table gives a summary of the work done in the year commencing July 1st, 1885, and ending June 30th, 1886, of each one of three hydraulic dredges at that time engaged on the work (exclusive of the months of January and February, during which time they did not work).

Much time was lost by reason of breakage, cleaning boilers, moving machine, arranging pontoons and discharge pipes, freshets, and the taking out of obstructions, such as logs and trees, that were frequently encountered in the bed of the river.

It is proper to state that at first the hands were generally green, and

Date.	Dredge No. 1.			Dredge No. 2.			Dredge No. 3.		
	Actual engine-hours.	Estimated cubic yards dredged.	Average cubic yards per engine-hour.	Actual engine-hours.	Estimated cubic yards dredged.	Average cubic yards per engine-hour.	Actual engine-hours.	Estimated cubic yards dredged.	Average cubic yards per engine-hour.
July, 1885	331¾	39 325	118	338½	94 157	278			
August, 1885	205	20 947	102	378	73 266	194			
September, 1885	231¼	38 271	165	224½	45 450	202			
October, 1885	453¾	97 800	216	359	59 375	165	214	25 232	118
November, 1885	341¾	62 282	182	367	60 159	164	330½	45 267	137
December, 1885	443	88 590	200	274	28 287	103	311½	48 348	155
March, 1886	342	76 303	223	303	41 250	136	405	79 842	197
April, 1886	374½	60 049	160	375½	48 287	129	304½	68 188	224
May, 1886	214	35 152	164	364½	50 685	139	267	58 017	217
June, 1886	370	72 442	196	419½	75 329	180	*182	34 772	191
Totals	3 307	591 131	179	3 403½	576 245	169	2 014½	359 666	179

Note.—The number of cubic yards dredged is taken from the inspectors' estimates as measured *in situ*, and is somewhat less than the actual amount.

much was to be learned from practical experience before the best results could be obtained with the least expense. The proper management of the suction pipe was a matter of no small moment and called into play the best skill and judgment of the operator. In order to do good work it is necessary that the hood at the bottom of the suction pipe be kept constantly at the proper distance from the bed of the river. If it gets too close, too much solid matter is drawn into the pump and may choke it; if not kept near enough, only water may be pumped through the pipes. A vacuum gauge connecting with the pipe is kept before the eyes of the man who regulates the suction pipe, and, by knowing the degree of rarification of the air in it, he is able to control its action. It is an operation requiring skill.

By means of the hydraulic method 5 036 651 cu. yds. of material were dredged from the river-bed and deposited on the flats.

The prices ranged from 12.37 to 15.45 cents per cubic yard.

Measurement was made *in situ* in the river-bed generally, but 1 000 000 cu. yds. were measured by cross-section on the fill.

One of the defects of the Von Schmidt hydraulic dredge is the rigidity of the suction pipe. It is telescopic, but has no flexibility, and even in moderately rough water it cannot be used, as the rolling

* This dredge was burned June 19th, after having worked .65 of that month.

of the vessel is almost sure to break the pipe. For this reason it is only applicable in localities where the water is comparatively smooth.

Another defect is an unnecessary number of bends in the suction and discharge pipe. It is well known that in forcing water through pipes a considerable loss of velocity is sustained at the bends, and that the number of them should be as few as practicable. The number of sharp bends in the pipes of the Von Schmidt machine are unnecessarily great, there being no less than three that could be avoided by a different arrangement.

Still another defect is the want of some appliance to remove logs, trees, etc., that are so frequently encountered in dredging operations. Some of these machines are provided with a small derrick, to pull them out after they are found; but the trouble is in finding them before mischief is done to the machine. They are a great source of trouble and delay when met, and not infrequently cause a breakdown before their presence becomes known. In some places these and similar obstructions buried in the soil are quite numerous, and some appliances should be on hand to detect and remove them. Or, if they be encountered unexpectedly, the suction pipe and the cutters on the end of it should be so arranged that it would not cause damage to the machine.

Another method for putting the spoils of dredging ashore, and used to some extent on the Potomac Flats, was by means of the Riker pump.

By this method the material was raised and discharged into the high end of a long chute, down which it flowed under the influence of gravity and the impulse given to it as it was forced out of the discharge pipe. The material was dredged from the Washington Channel and conveyed in bottom-dumping scows to the pump, which was located close up to the embankment previously constructed.

The pump was a steel cylinder, 7 ft. in diameter and about 30 ft. long. This cylinder sat in a jacket, about 9 ft. in diameter, giving a space of 1 ft. between the two, which space was allowed to fill with water. Near the bottom of the cylinder was an intake or suction pipe passing out horizontally through the jacket. This intake, 36 ins. in diameter, was provided with a flap valve that worked inward. Opposite to the intake was a discharge pipe of the same diameter, fitted with a similar valve that worked outward. This discharge pipe, after passing through the outer jacket, curved upward and was extended to

the high end of a chute, about 28 ft. above the water level. The pump was set in a hole nearly 30 ft. deep and braced to a surrounding platform built on piles. The mouth of the intake was thus nearly 30 ft. below the surface of the water. The material to be forced up into the chute was dumped into this hole over the mouth of the intake. Steam from the boilers, being admitted to the cylinder under a pressure of about 40 lbs., forced out through the discharge pipe whatever was in the pump. The steam being then cut off, a shower of cold water from a tank on top of the pump created a partial vacuum, and an inrush of material at the intake ensued. The mouth of the latter being buried in mud, it filled the cylinder.

Steam was then again admitted and the contents of the pump forced out as before, and the operation continued in a succession of pulsations at intervals of about one minute. The amount of material discharged at one pulsation depended on the amount within effective reach of the intake. It was estimated that about 1 500 cu. yds. could be pumped per hour if material was supplied with sufficient rapidity.

The ability of this pump to handle heavy objects was demonstrated on two occasions: at one time a stone, supposed to be a buoy anchor, weighing 1 300 lbs., and at another time, an iron safe, 25 x 16 x 14 ins., were forced through it.

The chute into which the material was discharged was a large frame-trough supported on trestles, and was about 456 ft. long. It had an inclination of 1 vertical to 25 base. The momentum acquired by the material as it was forced out of the discharge pipe, aided by the force of gravity, was sufficient to carry it to the end of the chute or some distance beyond. It was necessary, however, to keep the machine in nearly constant operation, and to pump large quantities of water, so as to keep the material on the dump in a jelly-like mass so it would spread out. A stoppage of work for a few days would cause it to take a "set," and, after that, the accumulation at the end of the chute would become so great that a change in the location would be necessary.

About 400 000 cu. yds. of spoils from the Washington Channel were put ashore by this machine at a cost, inclusive of dredging, of 13 cents per cubic yard, place measurement.

Another contract was made at a subsequent date for depositing 500 000 cu. yds. by the same method, on another part of the flats, at 14½ cents per cubic yard, but the contract was not fully carried out.

Much trouble arose from the material piling up at the end of the chute, which was caused chiefly by inability to keep up a constant supply.

This method of transporting spoils from the bed of the river was not as advantageous, in many respects, as the hydraulic. The construction of long chutes was expensive; the material did not spread out so well, and the fill was longer in becoming solidified. It is believed that the construction of the chute could have been avoided by using iron pipes. Large quantities of water had to be pumped anyway, and if the pipes had been extended on the fill as it was made, in a manner similar to that practiced in the hydraulic method, the level could have been kept down to the required grade, as the position of the end of the discharge pipe could easily and frequently have been changed.

The total amount of material deposited on the flats up to January 1st, 1893, was as follows:

	Cubic Yards.
By use of ordinary dredges and railroads on trestles..	1 418 771
By ordinary dredges and Riker pump.....	781 118
By ordinary dredges, but mostly taken in close proximity to, and for the purpose of forming, embankments.....	2 200 983
By hydraulic dredges.....	5 036 651
Total.....	9 437 523
More than three-fourths of the above was measured in place. Reducing all to scow measurement, the total would be about.....	10 800 000

The average price all the way through was somewhat less than 15 cents per cubic yard. The average price paid for the hydraulic dredging, which was the most satisfactory, all things considered, was about 14 cents per cubic yard, *situ* measurement. It is difficult to say just what scow measurement of the same material would have amounted to, but it is seldom less than 20% in excess of measurement in place. Converting place measurement into scow measurement on this basis, the price for the hydraulic dredging was only about 11½ cents per cubic yard, scow measurement, which price includes the profit of the contractor. This was but little more than one-half the price paid for the same work done by means of scows and railroads on trestles, the

amount invested in plant in each case being nearly the same for the same amount of work per month accomplished.

From the above it will be seen that the hydraulic method of filling low grounds and marshes is an economical one in many places, and is well worth considering by any one having such work to do. Where the improvement of navigation can be accomplished at the same time, this method has many advantages to recommend it. If it be not applicable for any reason, as, for instance, where the chain of pipes would cross a much-used channel, and they could not be laid on the bottom, it may pay to use one of these machines to put the material, dredged by other means, ashore. Material dredged by the ordinary clam-shell dipper or endless chain method is so cut up and loosened that the capacity of the hydraulic machine for pumping it ashore within a diked space is vastly increased. It is not to be understood that any claim is made that the Von Schmidt hydraulic dredges possess inherent advantages over others. There are other types of similar machines which may do better work. The work done by them on the Potomac Flats was satisfactory, and if others possess superior advantages, it is only an additional reason for using the hydraulic method of dredging wherever it can be practically applied.

Already in many places convenient dumping grounds for the spoils of dredging are becoming scarce. Where the towage exceeds moderate limits, the cost of this item becomes a heavy one, while almost invariably where channels are to be improved there are to be found low marshy grounds near by which would also be improved if the dredged material could be put on them. Heretofore the cost of doing this has been prohibitory. Dumping grounds are sought in the near vicinity of the work, from motives of economy alone. This has caused many places to be used for the purpose which should not be, for the material often finds its way back into the improved channels, or into others which have become impaired thereby.

The filling of marshes in proximity to cities is frequently a desirable thing from a sanitary point of view, while, aside from that, the enhanced value of the land created will, in some cases, more than pay for the cost. At Washington, the value of the land reclaimed is estimated to be worth more than the cost of reclamation.

The amount of settlement that takes place in material of this kind used for filling low places would be interesting if it could be accurately

determined. It was found on the Potomac Flats to vary very much. With a moderate quantity of sand in it, the settlement amounted to little, but where it was pure mud it was considerable and long continued. Material taken from the Washington Channel in 1881, and deposited by means of cars on trestles, had, up to 1884, settled some 2 or 3 ft., but the exact amount is not known. From 1884 to 1891 it settled from 1 to 1½ ft. more. It is safe to say that the settlement on this part of the flats has not been less than 4 ft. in some places in 10 years. The piles of the tramway used for transporting the material remained in a vertical position until some time after the work stopped, but in less than two years from that time they had been pushed over until, in some cases, they were several feet off the line. The material used in making this fill was very soft, and the causeway of Long Bridge held it on the west side. Moreover, the fill was made over the bed of the old swash channel, which crossed diagonally the line of this causeway; consequently, the bed of the river at that place was of recent formation.

At another place below Long Bridge the settlement, as determined by levels carefully taken, varied from .05 to .37 in. A remarkable fact in connection therewith was that the settlement of the material carried with it the piles on which the trestle had been built. The material used in this part of the flats was far better than the other. Some of it was gravel, and most of it was more or less sandy. No record was taken of the penetration of the piles, but it is believed that it did not in the last few blows exceed 1 in. with 1-ton hammer falling 15 to 20 ft.

Reference to the map herewith will show that by the filling of the flats according to the plans adopted, the Washington Channel would be closed at the upper end, making it an arm of the river in which the tide would merely rise and fall, but in which there would be no tendency to clear itself of the sewage, street washings and other refuse that would inevitably get into it. With a view, therefore, of clearing it of this source of pollution, but with no intention of flushing it in the true sense of the term, a large reservoir was designed to occupy a part of the space between Long Bridge and the sewer canal, which reservoir was to receive water on the flood tide from the Virginia Channel and discharge it on the ebb into the head of the Washington Channel, this inflow and outflow to be regulated by means of gates to work automatically, the plans for accomplishing it being left to the engineer to work out. To carry out this idea, a reservoir, excavated to a depth of

8 ft. below mean low tide, was established on Section II. Its area was about 110 acres; as actually laid out it is 111.17 acres. The depth of 8 ft. was fixed upon arbitrarily, but chiefly for the purpose of preventing the growth of eel grass on the bottom (see Plate V).

The mean rise and fall of the tides at Washington is 3 ft., but it is more or less influenced by the winds and the stage of the river in the fluvial portion above the Falls. A rise or fall of 5 ft. is no infrequent occurrence. The mean duration of flood is about five and one-half hours, and of ebb about six and three-quarter hours; but this is also influenced by the stage of the river above. Water being admitted freely at the inlet, the reservoir would receive about 14 500 000 cu. ft. on the average flood tide, or about 29 000 000 cu. ft. daily, enough to replace nearly one-half the entire volume of water in the Washington Channel. This, it was believed, would keep the latter fresh. The reservoir was to be provided with inlet gates, to admit the water from the Virginia Channel on the flood tide, but to close automatically on the ebb, and outlet gates to discharge it into the Washington Channel on the ebb, but to close in the same way on the flood.

After a careful study of the matter during the progress of the work it was found that there was a reasonable probability that the inlet gates might be dispensed with altogether and the connection made with the Virginia Channel by means of an open passage or canal. The fluvial discharge of the Potomac during a large portion of the year is considerable, and it was hoped that this would be sufficient to keep the level of the river at the site of the inlet above that at the outlet, in which case there would be no tendency for the water that entered the reservoir to flow back into the Virginia Channel. It was accordingly decided to build the outlet first and let the necessity of the inlet gates be determined by experience. The wisdom of this was fully justified. The outlet alone has accomplished more than was expected of it.

An examination of the site of the outlet, made during the winter of 1887, showed that, to a depth of 72 to 75 ft. below the level of mean low tide, the soil was a soft, sticky black mud, with an occasional layer below the depth of 45 ft. of sand and mud mixed; that at 72 to 75 ft. there was compact gravel or hard pan, and this was not far from rock. As the structure was to be a heavy one, and inasmuch as any unequal settlement would have been disastrous, it was determined to build it on a pile-and-grillage foundation, the piles being driven to the hard

material, so that they would act in a measure as pillars supporting the weight above. This weight amounted to about 10 tons per pile, the latter being spaced at intervals of 3 and 3½ ft. It is well known that this is not an excessive weight to put on a pile. They are frequently loaded to two or three times that amount, but in this case it was regarded as absolutely necessary that there should be no unequal settlement, and, in order to make sure of this, it was deemed best to found the structure so that, as nearly as practicable, there would be no settlement at all.

The soft nature of the underlying strata was not the only bad feat ure. In taking the borings marsh gas in considerable quantities was encountered. This gas seemed to be in pockets, and at various depths—from 25 to 65 ft. below the level of mean low tide. At one place when the well pipe struck the gas, the mud and contents of the pipe were thrown out of it with a noise like the discharge of a gun, causing considerable consternation among the men. The gas continued to blow off for hours, resembling the blowing off of steam from a boiler under pressure. The gas was inflammable; in one case it burned for 24 hours continuously; at first, with a flame 8 or 10 ft. long. Gradually the pressure diminished, and finally the flow ceased altogether. It was not believed that this gas would have any deleterious effect on the life of the foundation piles, as other piles used in the bridge, taken up from the ground near by, and where there was every reason to suppose the same conditions existed, were in a perfect state of preservation, though they had been down for at least 25 years.

The reservoir outlet, or sluice gates (Plate VII), as they were designated in the earlier reports, consisted of a masonry structure, or dam, with six arched openings through which the discharge was effected, and connected to the adjacent fill on either side by wing walls. The main head walls of the structure were built of granite, the wing walls of concrete. The faces of the former were vertical, and those of the latter had a batter of 12 on 1. The openings through which the water passes are 6 ft. wide, and 12½ ft. high to the crown of the arch. The gates, which allow the flow to take place outward only, are in pairs, pivoted on vertical axes, and, when open, swing into recesses in the masonry of the side walls. When closed, they butt like the gates of a canal lock on mitres at the bottom and top. Being pivoted at the bottom, and held in a vertical position by a hinge at the top, and always

partially submerged, the friction is so little that the slightest pressure one way or the other, moves them. When the tide begins to rise at the head of the Washington Channel, the gates come out from the recesses and close automatically, shutting off the passage of water from that channel into the reservoir. As soon as the tide begins to fall they open and allow the water to pass from the reservoir to the Washington Channel. The gates need no apparatus to start them from their recesses. They do so automatically, and have done so for several years.

The concrete used in the foundation of the main structure, as well as that in the wing walls themselves, was made of 1 part, by measure, of Portland cement, 2 of sand, 3 of pebbles, and 4 of broken stone.

In order to give a smooth surface to the faces of the walls, the stone was omitted on that part to a depth of about 3 ins., and the quantity of pebbles somewhat reduced. The concrete was rammed in layers of about 10 ins., and, in order to insure good bond between the layers, the top surface of each had spread on it with a brush a thin layer of mortar composed of 1 part cement and 1½ parts sand, immediately before the new one was put on.

The foundation piles of the main structure were cut off at a level of about 12 ft. below low tide and capped by 12-in. timbers secured by notches 2 ins. deep, and drift bolts. A second set of 12-in. timbers crossed the first at right angles, and the spaces between them, as well as the spaces around the heads of the piles for a depth of 2 ft., were filled in with concrete.

The pile and grillage foundation was constructed within a coffer-dam. The dimensions of the latter were fixed at 100 x 80 ft., in order to allow of the construction at the same time of about 30 ft. of each wing wall.

The depth of water at the site was from 4 to 6 ft., and, as the piles were to be cut off at a level of about 12 ft. below low tide, a dredge was sent in, to excavate a part of the material before the coffer-dam was built. It is probable that it would have been better not to have done this. The excavation made by the dredge cost but a fraction of what it cost to take the material from inside the dam after the latter was built, but, on account of the soft nature of the bottom and the disturbance due to a dredge working in it, the material was so much softened

that the top strata became about like jelly and gave great trouble afterward in holding the dam.

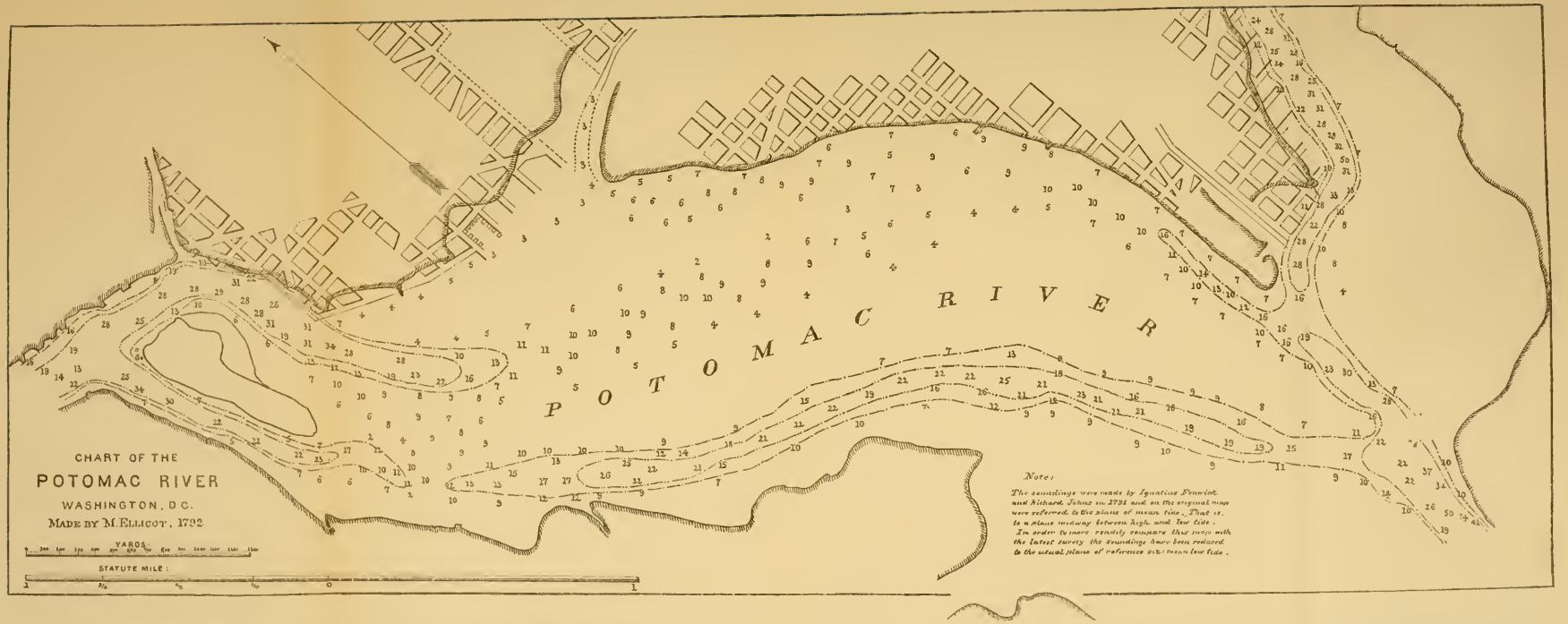
After the hole was dug by the dredge, a working platform was constructed on the north side and a tramway to it from the shore.

The driving of the foundation piles and the construction of the dam were then commenced and carried on simultaneously. The dam was built by driving two rows of piles 8 ft. apart, the piles of each row being 3 ft. apart. To these were fastened the wale pieces, one near the surface of the river-bed, the other near the top of the dam. The sheet piling was 3-in. pine, tongued and grooved, and driven about 18 ft. into the mud. Stay rods 1 in. in diameter were passed through each alternate pair of piles, and held in place by washers. The space between the sheet piling was then filled with clay puddle. Within the interior of the dam, a second row of piles was driven at a distance of about 12 ft. from the long sides, to help support a working platform and to stiffen the coffer-dam which was braced against them.

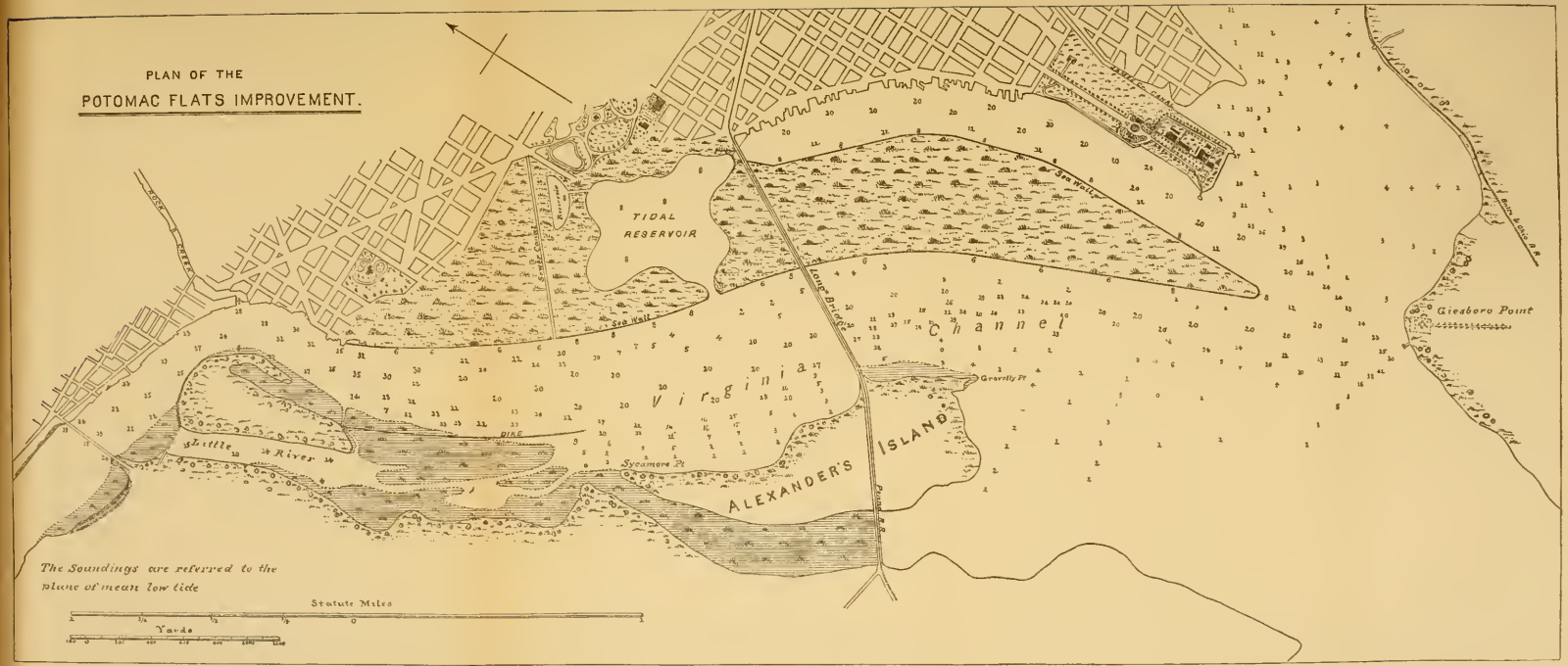
The dam was pumped out in 12 hours by means of a 6-in. centrifugal pump, but, as soon as this was accomplished, the four sides began to bulge inward, the corners remaining fixed. The movement was slow, but it was found to be continuous. The north and east sides soon came in at the middle about 2 ft. The south and west sides moved less, but all moved inward to a considerable extent, threatening the destruction of the dam. The question then arose whether to allow the dam to fill again and strengthen it before pumping it out a second time, or to try to hold it as it was. It was thought that by bracing the sides from one to the other on the interior, it could be held until the foundation piles were cut off and capped. Struts were accordingly put in from the east to the west side, and the north and south ends were braced by diagonals. After the dam was strengthened by bracing, the excavation of the interior began, but the bulging of the sides continued.

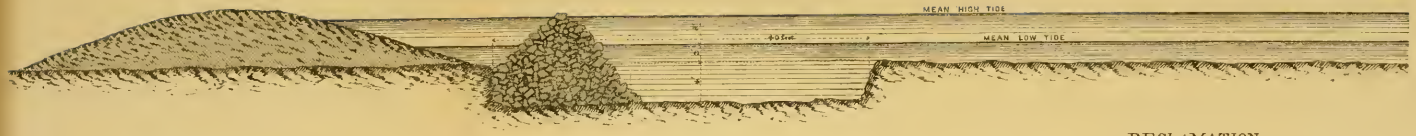
This developed leaks, though at first the dam was comparatively dry. Additional puddle was applied where needful, to supply what sank into the mud; but in some places it sank as fast as it was added, the sheet piles spreading at the bottom. Notwithstanding all efforts to hold the dam, it continued to move inward slowly.

It was then determined to drive a row of piles as close together as possible along the outside of the east and north faces. These piles were 60 ft. long and were driven till the tops almost touched water. A


POTOMAC RIVER
CHART OF THE
POTOMAC RIVER
WASHINGTON, D.C.
MADE BY M. ELLICOT, 1792
YARDS
STATUTE MILE :
Note:
The soundings were made by Ignatius Fenwick and Richard Johns in 1792 and on the original map were referred to the plane of mean tide. That is, to a plane midway between high and low tide.
In order to more readily compare that map with the latest survey the soundings have been reduced to the usual plane of reference viz: mean low tide.

MAP OF
POTOMAC RIVER
AT WASHINGTON, D.C.
BEFORE IMPROVEMENT WAS BEGUN
Survey of 1881
POTOMAC RIVER
ALEXANDER'S Id.
Easbys Pt.
Gieeboro Point
Hunters Point
The Soundings are referred to the plane of mean low tide
Statute Miles:
Yards:

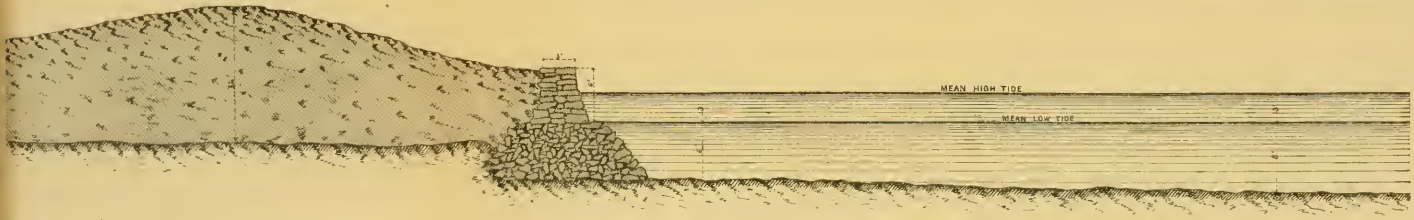

PLAN OF THE
POTOMAC FLATS IMPROVEMENT.
TIDAL RESERVOIR
Sea Wall
Sea Wall
Long Bridge
Channel
Virginia
Giesboro Point
Gravelly Pt
ISLAND
ALEXANDER'S
Sycamore Pt
Little River
DIKE
The Soundings are referred to the plane of mean low tide
Statute Miles
Yards



SECTION THROUGH FIRST CUT, WITH STONE FOOTING IN TRENCH.

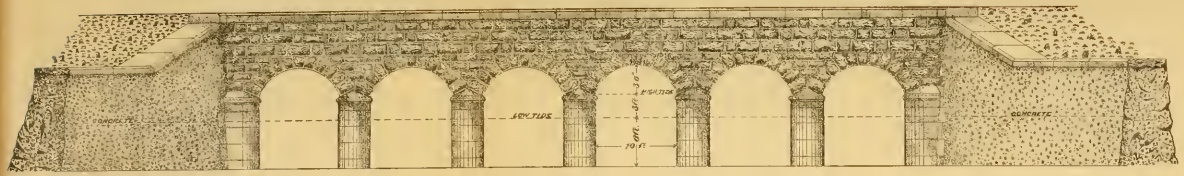
RECLAMATION OF POTOMAC FLATS.

HIGH EMBANKMENT WITH STONE FOOTING.

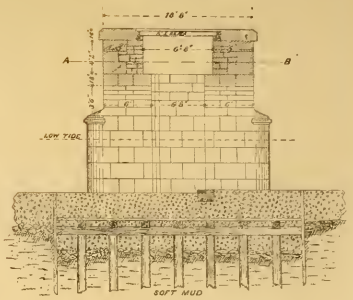


Scale 1 to 100

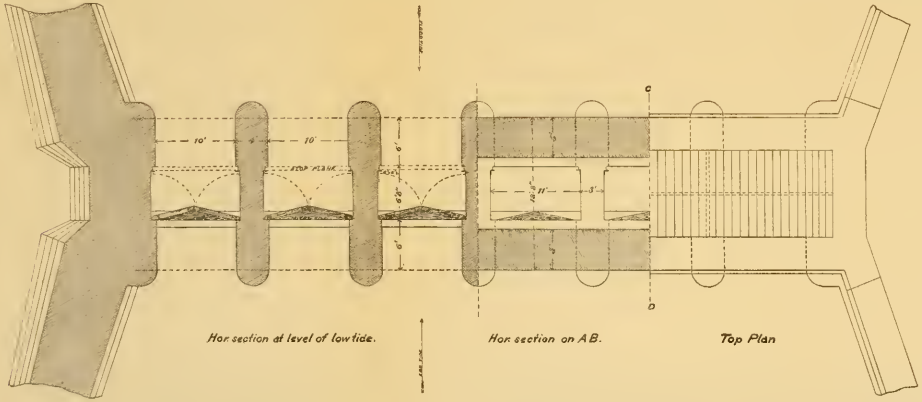
SECTION THROUGH FINISHED WALL



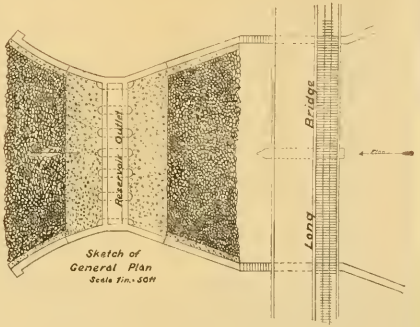
Front Elevation



Vertical Section on C D
gate removed.



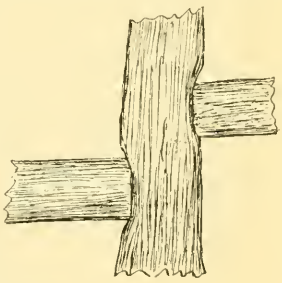
Hor. section at level of low tide. Hor. section on A B. Top Plan



Sketch of
General Plan
Scale 1 in. = 50 ft

second row was afterward driven along the inside of the same faces of the dam. More bracing was also added, and the movement was finally arrested. The excavation which had been begun was quickly finished. The piles were cut off and capped with the grillage timbers. When this was done, all danger was over, as there was then something to make fast to; whereas, before, everything seemed to be in motion. Measurements taken showed that the north side of the dam came in at the middle about 6 ft. and the east side considerably more.

Some of the strut timbers were pressed into the piles several inches, and in two cases where struts abutted on opposite sides of the same pile, but one slightly above the other, as in the annexed sketch, the pile between the ends of the struts was sheared off.



After the foundation piles were capped, no further difficulty was experienced in holding the dam. But the treacherous nature of the bottom indicated that great care would have to be taken with the wing walls so that they would not be forced in when the earth filling was put in behind them.

The top of the wing walls where they join the head walls of the outlet are of the same height as the latter, and from there slope off to the level of 6 ft. above mean low tide, which is the general height of the protective wall of the fill. After the experience had in putting in the foundation of the outlet, it was determined to construct an apron of concrete about 4 ft. thick, on a pile-and-grillage foundation extending from wing wall to wing wall within the entire space enclosed by the dam, and to cut off the latter at the level of the surface of this concrete, leaving all that part of the dam below that surface in the ground. The concrete wing walls were afterward extended, giving them a length of about 100 ft. on the up-stream and about 80 ft. on the down-stream side of the outlet.

In order to brace the foot of the wall against the pressure of the earth fill behind them and to more nearly balance the pressures on the two sides, the space between them was excavated to a depth of about 10 ft. and loose stone thrown in, forming a stone floor about 4 ft. thick. This appears to have answered the purpose very well; there has been no tendency of the walls to bulge inward thus far, and by

filling in behind them, gradually, it is believed there will be no trouble from this source.

After the reservoir outlet was constructed, the Baltimore and Potomac Railroad Company built a bridge within about 120 ft. of it, to take the place of a part of the old Long Bridge that spans the Washington Channel. The bridge consisted of two plate-girder spans of about 80 ft. each, resting on one pier and two abutments.

The wing walls of the abutments extended on the up-stream side so as to connect with the wing walls of the reservoir outlet, and on the down-stream side they were to unite with the sea wall protecting the filled area. The abutments were well built of brown-stone masonry resting on a pile-and-grillage foundation. On filling the approaches with earth, the south abutment settled 18 ins., while the down-stream wing wall moved out about 4 ft. This fact is referred to, to show that great care was necessary in the construction of a heavy structure like the reservoir outlet in that treacherous soil, and as a justification of the apparently extraordinary precautions taken to insure its stability.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

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690.

(Vol. XXXI.—January, 1894.)

RAILROAD LOCATION.

By MICHAEL L. LYNCH, M. Am. Soc. C. E.

READ OCTOBER 18TH, 1893.

WITH DISCUSSION.

The subject of railroad location from the nature of the case is rather a difficult one to handle through the medium of a professional paper, as it will admit of no fixed or inflexible rules, and it would be manifestly unwise to lay down any such for the guidance of an engineer on that class of work. The writer believes that a discussion of the general principles involved in connection with a résumé of the best modern practice would be of more practical value than any other way in which the subject could be treated.

Reconnaissance.—In the following discussion, a case is assumed where it is desired to connect with a line of railroad two cities, say 250 miles apart, with several intervening towns between them, which it is necessary or desirable that the road should pass through.

The first thing in order is evidently a reconnaissance of the country between the objective points, as a preparation for which the engineer should procure the best general map of the country available, together with such detail county and township maps as can be procured.

A study of the general map will usually show the existence of two or more possible routes, besides affording a knowledge of the main drainage of the country, which latter it is very necessary the engineer should become thoroughly familiar with, as also the tributary drainage as the work progresses. From such an examination he can also approximately determine where he may expect the steepest gradients, as the divide between two main drainages is never equidistant between the streams, which is readily determined by the location of the heads of the tributary drainage and their distance from the main stream or river into which they discharge; the nearer the divide to one of the streams, of course the steeper the slope on that side. In this connection the writer desires to draw attention to a peculiarity that has come under his observation and which he has never seen noted, to wit:

I.—In a stream flowing east or west through a country, the south slope of the valley is generally steeper, rougher, and more broken than the north slope, the latter being the smoother slope and with a gentler ascent out of the valley going north than descending into it from the south.

II.—In a stream flowing north and south, the east slope of the valley is invariably steeper, rougher, and more broken than the west slope, which latter is the smoother and affords a gentler ascent out of the valley going west than descending into it from the east.

In other words, the engineer will find it more difficult to get a line down into a valley from the south in the first case than in leaving the valley going up the north slope; the same applies to an east and west line in the second case. He will find it steeper and more difficult in getting down the east slope of the valley than in getting up the west slope, and where a line follows the valley of a stream, as in the first case, the smoother and better ground will invariably be found along the north bank; and along the west bank, in the second case, where a line occupies the slope or valley of a stream running north and south. A slight effort of memory will no doubt tend to remind any one of the existence of the above general law, it being of course more marked in some instances than in others, and being less noticeable where the streams in their general direction depart from an east and west and north and south course. It applies even on a large scale, as the Pacific slope is much steeper, more abrupt, rugged and broken than the eastern shore of the Continent.

There is possibly some geological reason why this condition should exist, probably something connected with the rotation of the earth on its axis during the formative state, which the writer will leave scientists to explain; he mentions the fact in passing, as having come under his observation and being worthy of note by the locating engineer.

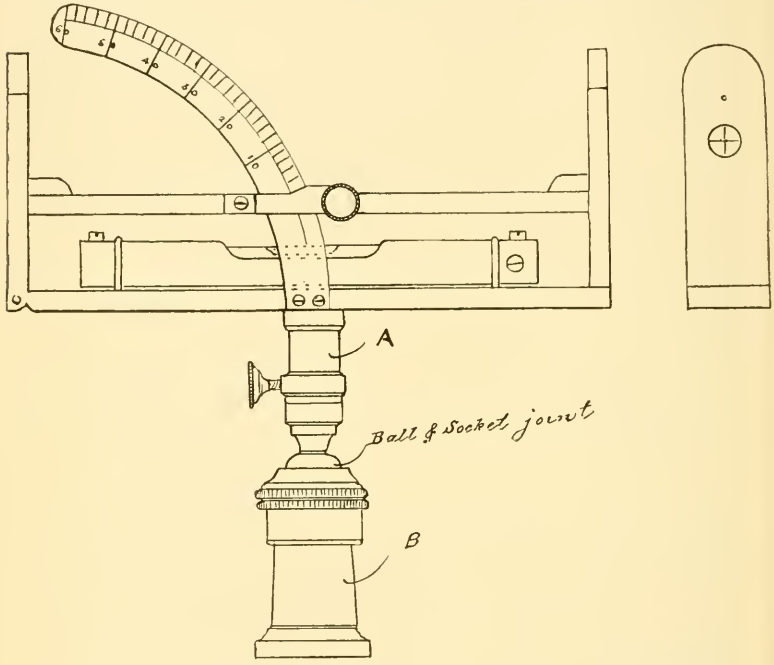
The manner of proceeding with the actual work of the reconnaissance will, to a certain extent, depend on the nature of the country operated over. A well-gaited saddle horse will be a great advantage, as much of the work can be done on horseback and a large scope of country rapidly examined. It will, of course, be necessary for the engineer to dismount and carry a line through on foot where very broken ground, dense brush or other obstacles interfere, leaving his horse to be led around by a man employed to accompany him for that purpose.

The usual portable instruments will of course be necessary in such an examination, but much will depend on the engineer's skill and familiarity with their use, as also on his natural judgment of country. This faculty, as is well known, is possessed by some men to a very marked degree; it enables them to keep track of their position with reference to the country previously passed over and to memorize its general features to such an extent that, after such an examination, the mind seems to absorb such a general knowledge of the country, its drainage and general topography, that a mental map of it is available when desired.

The instruments carried on such a trip are usually the pocket compass, hand level and aneroid barometer. The latter is to be used with much care and judgment, otherwise, owing to changes of temperature and meteorological conditions generally, the results obtained would sometimes apparently make the hills lower than the valleys, and water to run up hill; two aneroids accurately rated and compared, and read at the same hours by different observers within reasonable distances apart in settled weather, will usually give reasonably accurate results as to the difference of elevation between the points where the readings were taken, or the relative elevations of two or more gaps or depressions in a ridge can be determined with reasonable accuracy in settled weather by a single instrument in the hands of a skillful observer.

As a rule the determining of the feasibility of surmounting certain

difficulties at governing points, such as getting over a certain ridge and down the other side on a practicable gradient, is where the leveling instrument is most needed. The writer uses and recommends for that purpose a small hand level as per accompanying sketch, with sights like



a surveyor's compass, and with graduated vertical arc which can be set to any gradient desired, and will readily determine whether the maximum gradient is practicable along a given slope, and what the approximate depth of summit cut would be. Such an instrument would probably have to be made to order, as it is not catalogued or illustrated in any of the instrument-makers' manuals with which the writer is familiar; the ordinary hand level is generally used on this kind of work, and in the hands of a skillful observer will, of course, give good results.

In a reconnaissance of the kind under consideration, several governing points will present themselves—the crossing of certain ridges and of important streams, and the touching at certain towns. As a rule it will be found advisable to determine the lowest saddle or gap in the different ridges and the best crossing of the large streams in the gen-

eral direction of the line, together with the most suitable entrances and exits from the towns, the location of the various intervening sections of line being laid with reference to these governing points, and being more or less subordinate to them.

The reconnaissance being made between the two objective points on one of the possible routes selected, it will be advisable to examine a return line by another route for the purpose of comparison, and, in broken or very difficult country, to examine several routes in order to acquire a thorough knowledge of the country, as portions of the various lines examined may be combined and adopted on the final location.

Sometimes a good line can be had along a ridge where it is wide and flat and the minor drainages starting from it head some distance on either side, away from the divide; at other times a ridge line will be found impracticable owing to its narrowness and generally broken and crooked character, caused by the minor drainages heading on either side of the general trend of the divide.

The writer would recommend the making of copious notes and sketches during the progress of each day's work; this necessity will become quite apparent when the actual work of locating is undertaken.

The result of a thorough reconnaissance as above outlined, when carefully platted and sketched in, will show such a practically correct and useful topographical map of the country as will be quite a surprise to one not familiar with this class of work, as the lines run with a hand compass and the distances determined by timing a well-gaited horse give much more accurate results than one would ordinarily suppose to be attainable under such circumstances.

In this connection the writer desires to draw attention to the importance of a thorough reconnaissance before starting the instrumental preliminary and location surveys; otherwise the best line in a country may be overlooked and an inferior and more difficult general route adopted, which, no matter how skillfully the actual detail work of location might be performed, would be inferior to the omitted line for purposes of railroad operation.

Preliminary Survey and Final Location.—The reconnaissance being complete, the next thing in order would be the organization of a party or parties to take the field and run the preliminary lines and the final location.

The general organization of such parties is usually similar, but may differ somewhat in detail in different localities.

In thickly settled sections of country the reprehensible practice is sometimes adopted of starting the party out without a camp outfit, providing for their subsistence and accommodation at the different farmhouses along the line. This method will not be further referred to than to say that it is an arrangement which, at best, cannot be too strongly condemned for various reasons too obvious to need mention.

A party properly organized for preliminary or location survey in ordinary country should have a proper equipment of not less than three good teams and wagons, tents to accommodate the rank and file of the party, office tent for the engineers, and one to answer for a cooking and dining tent, with saddle horse for the locating engineer and one for the chief of party, although the latter is not indispensable. The party should be full-handed and include draughtsman and topographer, together with cook and teamsters; the teams in addition to moving camp, to be used in taking the men to and from their work on the line and taking lunch to them at noon.

It will be found good practice to transport the men to and from their work, as thereby more hours of work can be obtained from them and they will be fresher and in better condition to perform the work required of them.

In conducting the preliminary survey, it is, of course, desirable that the line should be run as close to where the final location would lay as possible, the chaining to be carefully done, and all angles turned off on the vernier of the transit; in other words the preliminary line should be accurately run as the location is built upon it, and inaccuracies in the preliminary lines are sure to cause trouble and annoyance when it comes to final location.

This also applies to the work of the topographer, as the accuracy with which the contours are laid, and streams, buildings, etc., located upon the preliminary map, has a direct bearing upon the case and the accuracy with which the final location can be made. The topographer will find the elevation of each station as obtained from the level notes of great advantage to him in properly laying on his contours, and with a judicious use of the tape line and slope level should be able to furnish an accurate topographical map for several hundred feet on either side of the line, including the general course of streams crossed, etc.

For such a survey as the one under consideration several preliminary and spur lines may have to be run, and while to the uninitiated the work might appear to be making slow progress, still it is, nevertheless, good practice to thoroughly develop the country by means of preliminary lines before deciding on the final location. This the engineer sometimes finds difficulty in doing, as, owing to the ignorance of those who should be supposed to know better, the attempt is often made to force him to establish a line without giving him the necessary time to properly develop the country, the result being that many roads are to-day suffering from the effects of mislocation, caused by crowding the engineer on his work, and failing to accord him the necessary time to properly develop the country. The evil effects of such a practice are continuous, in the increased cost of maintenance and in the operating expenses of moving trains over steeper gradients than there is any actual necessity for.

In this connection it may be well to say, that, counting the time occupied on preliminary and location surveys, wet days, etc., one mile a day of finished located line is a fair average in an ordinary country, partly timbered and partly open; more than this can of course be done in a smooth open country, and not so much in a very rough broken country.

The preliminary lines having been completed, and one of them, or a combination of several of them, having been decided upon as the most feasible route to adopt, the work of final location is proceeded with, the tangents and curves of the proposed location being first projected on the carefully prepared topographical map of the preliminary lines before laying off in the field. It often happens, however, that the location so projected can be improved, and the best practice, in the writer's opinion, is to recognize the possible necessity for such change and be prepared to make it in the field when necessary as the exigencies of the case demand; in other words, no cast-iron location can be projected on a preliminary map in the office, no matter how accurate such a map may ordinarily be, as such lines are liable to be susceptible of improvement in part when it comes to actually laying them on the ground. There is no royal road to railroad location. It cannot be done from a telegraph key, or a typewriter desk. To obtain the best results the work must be actually executed on the ground by one especially skilled in that branch of engineering.

A scale of 800 ft. to 1 in. is a convenient scale for preliminary maps upon which the location can be projected ordinarily. On difficult sections of line the writer has found it convenient to increase the scale for such portions to 400 ft. to 1 in., projecting the location on it, taking scale and protractor on the line, prepared to re-project and re-run the line until the best result obtainable in the country is attained. It will also be found useful to have a small strip of profile paper along, so that the level notes of some doubtful or difficult piece of line can be plotted as the work progresses. In general, the more experience the engineer has on this class of work, the more careful he is to take every advantage and precaution to get the best line the country affords, recognizing the fact that there is a best line in every country, and that it is his province and his duty to find it; and, strange as it may appear to the layman, the skillful locating engineer can tell unerringly when he has found the best line in a country.

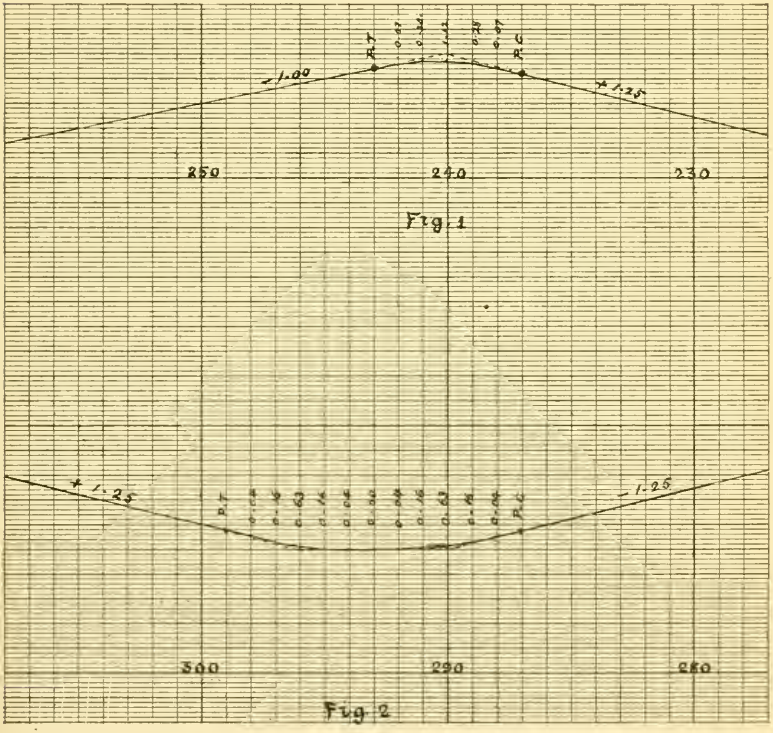
In regard to curvature and grades it is assumed that a certain maximum has been determined upon, which must not be exceeded. In this day of rapidly increasing speed it is desirable that no reverse curves should exist—a few stations, at least, of tangent between curves of different directions being a necessity to admit of the engine and leading cars of a train straightening themselves out before entering the curve ahead. Of course, the more of such tangent the better. Heretofore transition curves were rarely used, but, owing to the increased speed that now obtains, they may be considered a necessity where the curvature is at all sharp.

"Broken-back" curves, as they are called, should be avoided—that is to say, two curves in the same direction with a short stretch of tangent between them; they are "bad form," to say the least, and one curve covering the ground and eliminating the short stretch of tangent can usually be found with a little extra labor.

In compensating a gradient for curvature it is now generally conceded that a compensation of .04 ft. per station per degree of curve is about right, and at the junction of two gradients on a summit or the bottom of a depression, it is very necessary to round them off by vertical curves. The two gradients can be run together, as in Fig. 1, and a vertical curve run in connecting them by a simple calculation, as follows:

An ascending grade of 1.25%, Fig. 1, meets a descending grade of

1% on a summit. The ordinates of a curve can be calculated by taking one-half the sum of the rate of grade per station, regardless of algebraic signs, as the ordinate of correction at the apex, one-fourth of that ordinate as the correction at the first station on each side of the apex, and one-fourth of this last ordinate as the next correction, and so on.



For example, the ordinate at the apex, Station 240, would be $\frac{1.25 + 1.00}{2} = 1.12$, one-fourth of which $= 0.28$ would be the ordinate for Stations 239 and 241; one-fourth of this last ordinate $= 0.07$ would be the ordinate for Stations 238 and 242, after which Stations 237 and 243 could be assumed as the beginning and end of the curve. This would result in a curve 600 ft. in length, and will be found to give a good practical result in connecting two such gradients.

Fig. 2 is an example of two 1.25% descending gradients in connection with a level grade of six stations. By the same rule the ordinate for Stations 290 and 296 would be $\frac{1.25 + 0.00}{2} = .63$, and for the stations, on either side, .16 and .04, etc., making Stations 287 and 299 the beginning and end of curve which results in a vertical curve 1 200 ft. in length. The writer prefers a stretch of level grade connecting two descending grades, as in the last example, in preference to connecting the gradients direct, as in the case of the two ascending grades, Fig. 1. The calculations for establishing vertical curves in this manner are so simple that they can be performed mentally. The resulting curves have been found to give good practical results in operation.

It is desirable by all means that vertical curves should be run in at the foot of all steep gradients, their absence being a fruitful source of derailment at these dangerous points (generally over some opening), owing to the tendency of the rear cars of a train to bunch or crowd those in front where a suitable method of transition has not been provided for between two opposing gradients.

It will no doubt be quite apparent that some of the methods set forth above, especially in connection with reconnaissance, would be inapplicable or unsuitable in a country already covered by complete topographical surveys, such as the ordnance surveys of Great Britain, or the other elaborate government surveys of the older and more thickly settled European countries of which elaborate topographical maps are available, which materially lighten the labors of the engineer, and from which the location can be intelligently projected and the line approximately laid in the office. The system outlined has more particular reference to such countries as America, Russia and others of large extent, where extensive areas are yet uncovered by such elaborate topographical surveys.

In this paper the writer has endeavored to give his views in a general manner on this important subject, without attempting to lay down any inflexible rules, being aware of the fact that "circumstances alter cases."

DISCUSSION.

C. E. Moore, M. Am. Soc. C. E. (by letter).—The literature of railroad location is quite scanty. Especially is this true of that which has been written by practical men. For this reason a paper like that by Mr. Lynch is very welcome, and I have read the abstract with much interest.

As to the general principles, it would seem to be an excellent statement. In the matter of the use of preliminary maps and the actual location from them, I feel compelled to dissent when they are applied to difficult work. For a difficult country, the scientific location is a calculated one.

For locating in a mountainous country, on steep slopes, perhaps descending from a summit with long stretches of a maximum grade, "re-projecting and re-running lines to get the best" would be unsatisfactory. If the line is through thick forest or dense underbrush, it would be still more so.

On such work the experience of the writer is in favor of:

First.—An accurate preliminary, as near the final line as possible.

Second.—Careful topographical notes, carried as far out as necessary.

Third.—A map carefully platted to a scale of 50 ft. to 1 in.

Fourth.—A final line fitted on the map and drawn in.

Fifth.—The final line to be calculated with reference to the preliminary, so that notes can be given the locating party to run in.

Such a location will rarely require any modification afterwards.

If the location is on a maximum grade with compensation for curvature, the preliminary should be run from the summit down, with the same maximum grade and with compensation for angles turned.

The writer does not advance these ideas as new ones, but simply as a part of the subject treated.

W. B. Storey, Jr., M. Am. Soc. C. E. (by letter).—It may be of interest, in the discussion of Railroad Location, to note certain methods followed on the Pacific System of the Southern Pacific Company, under Mr. William Hood, Chief Engineer.

Examination of country and determination of routes for instrumental survey are similar to the methods usually followed, which are described by Mr. Lynch. The difference begins on the instrumental survey. The parties are organized as usual, except that the head chainman is the best-paid man in the party, outside of the instrument men, the rodman ranking next. Especial care is paid to chainage, both chainmen being required to use plumb bobs and both being supplied with hand levels for leveling chain. The chain is checked daily by a standard steel tape. The amount of error allowed is $\frac{2}{10}$ ft.

in 1 000 ft., or about 1 ft. per mile. This kind of work is done on all kinds of work, preliminary as well as location.

The transit work is given equal care. If running on a grade line, a line of check levels is carried ahead of the transit instead of behind, and the check leveler also runs a line of grade levels, getting his distances by pacing.

The preliminary is an angular line, run as near where the final location will be as it is possible for the engineer in charge of the field party to tell, using the line of grade levels and a compass as a guide. This requires an experienced locating engineer, and even then it is sometimes necessary to back up and try another way. On open country less backing up is necessary, provided always that the engineer has studied his country carefully before the line is run. The object aimed at is to have the preliminary as near as possible of the same length as the location and the same so far as angles are concerned, and to make it a working base for study of the ground with reference to location.

The notes of this carefully run preliminary line are platted on scales much larger than the usual practice, viz., from 50 ft. to 1 in. in mountain country, to 100 ft. to 1 in. where longer tangents are used, and in some cases to 400 ft. to 1 in. Extreme care is taken with this plat, the angles being laid off by means of tangent tables, the distances carefully checked and the stations with plus fifties pricked on. This plat is then turned over to the locating engineer, together with a profile of the line. No contour notes or other notes are placed on the plat, but simply the line.

Then comes another departure from usual methods. The engineer takes two men, with hand level, rod, chain and profile, and goes on the ground personally. He takes notes in the field of the data, which the engineer usually studies on the contour map. He notes also details, which in many cases seriously affect cost of work, but which are not shown on contour maps, for two reasons, viz., that the scale is too small, and that the man who took the contour notes did not know what should be taken. These notes taken by the locating engineer supply him with everything necessary to enable him to make a careful, close, final location. With his memory keeping in mind the entire country, he goes to work on his plat and determines the best place for the line. He draws his tangents and their connecting curves, or, as might be said in mountainous country, his curves and connecting tangents, about as is usually done, but with a much more intimate knowledge of his work than can possibly be obtained by usual methods.

Here comes another radical difference. The location is transferred from the paper to the ground, not by scaling, but by calculation. Each tangent is fixed, and the connecting curves are all calculated in the office by carrying the line from one fixed tangent around through

the preliminary line to the next tangent. These calculated notes are then given to a machine known as the locating party and put on the ground by it. Every half mile or so a check is given to be taken on the preliminary, and this check must come out within certain limits, or there is error either in the preliminary or location, and it must be looked up. A record of checks is kept and left in the office at night, and this affords means for the engineer to see how the men are doing their work, and to know that errors do not exist for the constructing engineer to pick up.

A trip over the line by the locating engineer completes the work. He then notes culverts and other necessary details and sees that everything is as planned.

Of course there are alternate lines to be run, and other matters connected with location are as usually done. The unique features of Mr. Hood's methods are the carefulness with which all the work is done, the system of checks by which error in the finished line is next to impossible, and the intimate knowledge of the work required of the locating engineer. The keynote of the thing is accuracy. It is probable that more miles of railroad have been located in this manner than by any other one system of location in the United States. The Sunset route from near Los Angeles, Cal., to the crossing of Devil's River, Texas, the line over the Tehachapi Mountains, in Southern California, and the line up the cañon of the Sacramento to Mount Shasta, thence over the Siskiyou Mountains into Oregon, are well-known examples.

There are, of course, many minor details connected with location that cannot be mentioned in a general paper. It will not be amiss, however, to mention one that I consider very important, and one that is neglected by nine-tenths of the locating engineers in this country, viz., the manner of staking the line. The rule with us is to use stakes about 18 ins. long, or longer in soft ground, and to drive them so that from 4 to 6 ins. remain out of the ground with from 12 to 14 ins. in the ground. It is too often the case that surveys costing thousands of dollars are worthless in a year or two after being made, and when the company is ready for construction the entire location has to be made over, because the locating engineer has not paid attention to the small matter of seeing that stakes were well driven.

Relative to reconnoissance, Mr. Lynch does not mention the use of the odometer. On our western deserts we substitute two good mules, a buckboard and an odometer for the horse spoken of by him, and obtain very close results.

In conclusion, I would say that the rules laid down by Mr. Lynch are seemingly expensive, and the methods described above even more so, but it will be found that they pay, and that by using them railroads can be built for much less money than by the old methods. I admit, however, that the item "location" will form a greater percentage of the total cost than formerly.

Howard G. Kelley, M. Am. Soc. C. E. (by letter).—The paper presented by Mr. Lynch appears to me to be a step in the right direction, and is a practical description of the work of a branch of the profession which it is impossible to learn from text books.

In considering that portion of the article where Mr. Lynch says that the south and east slopes of valleys are more rugged and steep than the north and west slopes, the most notable example will be the great Continental Divide, with its long, gradual descent from the mountains to the Mississippi Valley on the east, and the sharp and rugged descent from the summit of the mountains to the Pacific Coast on the west.

The writer has also compared it with his own local experience in different parts of the country, and, while a corroboration is somewhat obscure in many localities, there are some examples where the theory appears to be borne out; but so far as it applies to local conditions, the writer would be inclined to consider it more as a condition liable to exist and to be expected than as a general law of universal application.

On the Columbia River, from Wallula Junction to the mouth of the Willamette River, in Oregon, the south bank of the river presented the rougher and more rugged country along which to locate a line; the south bank of the Puyallup River, in the State of Washington, is also rougher than the north bank, or, in other words, the river hugs the south side of the valley. In the case of other rivers of the Northwest, breaking, as they do, through mountain gorges, the examples are more obscure.

In the case of the St. Louis Southwestern Railway, at the points of crossing of the White, Arkansas and Red rivers in Arkansas, the conformation is obscure, as the rivers flow southeasterly: but the right-hand bank of all of these rivers is the more rugged in that particular vicinity. In Texas, however, the crossings of the Trinity and Brazos rivers show the steeper slope to be on the east bank.

The experience of our locating engineers will bear out Mr. Lynch that a thorough and careful reconnoissance is indispensable for a good location whether the country be new and unmapped or thickly inhabited. In the case of the unmapped countries of our western territories, the work of the engineer on reconnoissance is one of privation and hardships, requiring the strongest physique and powers of endurance; it has usually been done by an engineer attended by one or two assistants or "packers," the blankets and provisions being carried on the back by the aid of pack-straps, and many of the engineers of this Society can look back a few years to periods of semi-starvation and terrible exposure among our western mountain ranges. In such examinations nothing but experience and the natural faculty of location will enable the engineer to form a correct opinion as to the line to be used and the distance traveled.

In the West, the mule or "cayuse" pack-train takes the place of the team and wagon of the less rugged and more thickly settled country, the usual load for a pack animal varying from 250 to 350 lbs., depending upon the character of the trails, although the writer was at one time a member of a party which was relieved with provisions late in the spring after a long winter's siege, the animals in this case being barely able to travel through the snow with but 50 lbs. of load to the animal; but the luxury of a frozen can of tomatoes, split open with an axe, was more keenly enjoyed that day than a banquet at Delmonico's would be now.

The writer's personal preference is for a map drawn to a scale of 400 ft. to 1 in., as it corresponds with the scale of the profile paper; but for especially rugged sections a scale of 100 ft. to 1 in., with contours for every 5 ft. in vertical height, would be recommended.

A convenient way of taking accurate contours for very difficult sections is to allow the topographer a helper or tapeman; the topographer uses a hand level, and the helper a light cross-section rod and 50-ft. tape; the leveler, as he levels over the line ahead of the topographer, has his rodman mark on the back of each stake its height above the datum plane to the nearest foot. The topographer knows the height of his own eye above his heel to the nearest tenth, and thus standing at the center stake for a start he can rapidly level up or down the side hill, carrying his height of eye (which corresponds to an instrument height) mentally; the distance out to each vertical 5 ft. being recorded in his topography book in a manner similar to the fractional form of a cross-section book, the vertical height being the numerator, and the distance out from the center line the denominator. A skillful topographer can in this way keep up with the party, and the day's record is rapidly transferred to the contour map in camp at night.

It will be surprising to those who have not used this method what an accurate profile can be made from the contour map upon which a trial location has been projected.

The writer's later experience on maintenance corroborates Mr. Lynch, that a stretch of level grade is advisable between two descending grades; he has observed that the cars are less liable to bunch or break in two (that is, become uncoupled while running), where grades are adjusted as above, and further, that derailments are more frequent where two sharp descending grades come together, with nothing but a short vertical curve between them.

It is to be hoped that this very interesting paper presented by Mr. Lynch will receive a thorough discussion, and that our locating engineers from different sections of the country will add to the information presented by him.

D. H. Ainsworth, M. Am. Soc. C. E. (by letter).—Mr. Lynch is right in saying that "fixed or inflexible rules for location cannot be

laid down," but does he believe, even then, in leaving, sufficient discretion to the engineer?

We will take for granted that the engineer cannot know too much of his subject; that he has learned all about the physical obstacles to be encountered; all about the drainage and elevations; that he has employed a skillful topographer, and can make in his office a profile of any probable line; that he knows the population of each township, and the important trade points, and has availed himself of every means of obtaining information, not fearing to be accused of a want of reliance upon his own "eye for country."

Then does he know who is behind him with means, and how much per mile he can rely upon being spent, and how much he can prove can be made to afford a reasonable return on investment? In other words, has he any right to do anything that will not pay either directly or indirectly, immediately or in the near future?

May he not do worse than the best, with a certainty that finally his road may not have to be rebuilt?

Has he a right to consider his own reputation as an engineer the one thing paramount, assuming that he undertakes nobody's dirty work, and that he has decided all ethical questions properly and on the side of honor and right; that he is employed by no mere subsidy hunter or adventurer working merely for what can be made out of mere construction, and that his employers intend to operate as a commercial enterprise the project he is engaged in?

If he pleases, let it be his own money he is to use which gives him full control, would he do differently than if using the means of somebody else? I have in mind an instance where I myself proposed a tunnel cut-off that would now be deemed a good engineering feat, but that the company properly objected to spending their money in what they deemed an unremunerative project. Again, an engineer resigned because, notwithstanding his protest, the advice of the consulting engineer was taken, and an increased distance of 35% was injected into the line. While I have no doubt about the validity of the engineer's objections, I always claimed that he need not have resigned, as, after his protest, the responsibility was off his shoulders, and shifted to those of others.

I once located a line as I thought it ought to be built, but showed how it might be cheapened by a change at one point. I had nothing to do with the construction, but what I deemed the poorer line was used because of this saving of expense, to my own mortification, and I concluded never again to show what might be done unless it was what ought to have been done. Was this right? Is it right for an engineer to use deceit in any way because he knows what use will be made of the truth? For instance, he locates a line, but finds he can throw out several of his cheapest miles—of course, a saving of both cost and

distance—but increases the cost per mile, which will be considered in spite of him. Is he not justified in keeping his first location to himself? This is no mere speculation, but has been part of more than one engineer's experience. A man starts to do his best work, but a president, or possibly a chief engineer, gives him a value per mile of saving of distance. Invariably such valuation is not adhered to, and fault is found with the engineer, and changes ordered because he has taken some one else's advice.

Many a western engineer has had his work criticised because it was not done in accordance with views adopted twenty years later, when, if those views had been entertained at the time, the work would never have been undertaken, or would have been postponed indefinitely, those spending the money in improvements, or, possibly, re-building, being quite facetious at the expense of the original.

I have in view a short railroad that has been in operation twelve years and never paid, because handicapped by the expense of a bridge, the cost of which took all the profits of the other 35 or 40 miles. Still, the locating engineer got credit for finding a place for such a structure, which was proof of his ability, and possibly the engineer who had studied for its avoidance would always have the reputation of a nobody, even if he got something profitable for his employers.

How many engineers will admit that any railroad is either a fraud or failure that does not now nor never will pay a dividend; that one that cannot compete with ox or mule trains has no right to existence?

Again, secrecy as well as haste is frequently a good qualification for an engineer. A "booming" city cannot be approached safely with a publication "in town meeting" of a company's designs. A company has a right to a policy which a real-estate dealer may not know, and it sometimes calls for a good deal of policy on the part of the engineer to avoid publicity.

I have tried stadia work successfully in additions to St. Paul, simply to keep from real-estate agents the knowledge that a railroad was likely to need some of his wares, by having a party so small that it was not suspected of belonging to a railroad company, but rather to a city or local engineer's force.

The late Col. F. U. Farquhar once expressed surprise that railroad engineers did not make more use of stadia measurements, and I think I have profited by the hint.

Mr. Lynch, having been in the wilds of Texas, certainly knows of the necessity of always carrying along a duplicate set of instruments, to be available in case of accident. With a transit with vertical limb and stadia hairs a second party can be put in the field, perhaps by the use of teamsters or any other supernumeraries, and a grade can be tested and a line run, that can be plotted, to test the practicability of any single plane. A preliminary could thus be secured over the whole of Mr. Lynch's 250 miles that will cost little money.

Perhaps at least half the distance requires only geographical knowledge to be obtained from a map. A spring wagon and four can thus be made useful in obtaining an accurate preliminary knowledge of the whole 250 miles. Of course I would not leave out the two barometers that have to be used with so much care in accomplishing good guess work.

I would insist, also, on having a good topographer, no matter how much salary was required to secure one. They are scarcer than they ought to be ; I mean such as can be trusted to get contours within 500 ft. of him with sufficient accuracy to make a good profile within those limits. My own experience has been to advertise for one, taking on trial each one that came, discharging as soon as incompetency was discovered, and securing a good one at any reasonable salary. My experience, too, has been against promotion, or trying to make something else of the man secured for this specific purpose.

C. D. Purdon, M. Am. Soc. C. E. (by letter).—A locating engineer is born, not made; without a natural aptitude for the work, a man is unlikely to be a successful locating engineer, study he never so much. Still, a discussion on the question is of value, for, although an engineer may have had large experience, the ideas of other engineers are of service, if they should only suggest a subject to think upon.

Many locating engineers do not seem to keep sufficiently in view the fact that a line has to be operated after it is constructed, and this may cause location, which, while seemingly economical, is, in reality, the reverse.

As a general rule, embankment is better to operate than excavation; still, it is common to see a line which follows a creek bottom carefully located so as to have the center line as nearly as possible at grade, and the cuttings just about sufficient to make the fills, giving a succession of short cuts and fills with a necessity for a bridge or culvert in each fill. A better location, from an operating point of view, would be to keep the line in the bottom on a moderate embankment, and collect the water to a fewer number of larger openings, reducing the number of bridges to be maintained and avoiding soft cuttings in rainy weather. The mere earthwork on a road is but a small part of the cost of construction, and it is poor policy to sacrifice grades and alignment to save a few hundred cubic yards per mile, especially to sacrifice alignment ; for, while grades can be improved, it is a different matter to improve alignment, except within narrow limits.

Sharp curves also, while they may reduce the quantity of earthwork, limit speed of trains and materially increase the wear of rails and wheels, besides having other disadvantages.

The economy in construction only occurs once ; the waste in operation is perpetual.

When a country is sectionized, the work of reconnaissance is much

assisted, as governing points can be easily located in advance by section corners. The information obtained from the average farmer, although his advice is always freely offered, is rather delusive ; he generally has a pet "plumb level route" which he is anxious to show, and his ideas of high water-marks, with best possible intentions, are of a vague nature ; a very common high water-mark is "deep enough to swim a horse."

I agree with Mr. Lynch as to its being a bad practice to survey without a camp outfit and sufficient teams ; besides the discomfort, enough time is generally lost to purchase several camps.

Speaking of the topographer, Mr. R. L. Van Sant, M. Am. Soc. C. E., employs a neat method of taking topography. The topographer keeps a day behind the party, and has the previous line plotted upon a piece of tracing linen, with the elevations of the different contours marked where they cross the line. Taking this tracing, with a scrap of cross-section paper of suitable scale under it, the contours, streams, etc., are plotted in the field with considerable accuracy, and very rapidly transferred to the map by laying the tracing over it.

It is rarely the case that an engineer gets sufficient time to make a location, the company being in a hurry for profile; it might be broadly stated that no engineer has yet made a location where he did not observe points capable of improvement on going over the line after construction.

The average of about one mile per day in average country, of completed location, agrees closely with my own experience. I think the average cost of location might be taken at $60 per mile.

Preliminary lines should have the instrumental work and chaining very carefully done ; and the map should be plotted by latitude and departure, retaining the calculated notes for use in projecting location and making field notes therefor. By doing so I have generally found that the located line came on the ground where it was projected, and many of the curves could be run directly without first running the tangents to intersection.

There is some difference of opinion as to the proper amount to reduce grades for curvature. I do not think it material whether the amount of reduction used be exactly correct ; if it is excessive, an engine will stall on the tangent ; if too little, on the curve.

Vertical curves in gradients should always be used. I have found Mr. Lynch's rule give good results, besides having the merit of being easily remembered.

As Mr. Lynch has had a very large experience in location, his remarks thereon deserve attention, and I know his methods to be thoroughly practical.

Mr. R. G. Tarr (by letter).—Mr. Lynch states that a divide is not equidistant between two streams, but is nearer one of them, and that the slope on this side is steeper. This follows as a necessary result in

the vast majority of cases, because the intensity of erosion is greater; in other words, the stream has more work to do in a short distance. The reason why the divide is at a greater distance from one stream than from another is too complicated a subject for discussion here. It depends upon geological conditions, often of great complexity, and also at times upon variations in rainfall upon the two sides.

There are, however, exceptions to this general statement, as, for instance, when a stream has had its normal development interrupted by some accident. This is the case in some of the northern streams, north of the terminal moraine, that is, north of a line passing through Long Island, Central New Jersey, Northwestern Pennsylvania, and, in general, westward to Madison, Wis., and thence northwestward. North of this line many of the streams have accidental courses, caused by dams of drift placed across their courses by the continental glacier. This statement is therefore not an absolutely safe one to follow.

With reference to the other points made by Mr. Lynch in this connection, namely, that the slope is steeper on the south side of an east and west valley, and on the east side of a north and south valley, I am convinced that he is in error. No such generalization can be made, and the case of the Pacific slope, which he mentions, is dependent upon geological conditions, as indeed is the entire subject of the position of divides and the slope of valleys. It is a complicated subject, and no single law can be stated which will not be found to have innumerable exceptions; and many exceptions to the generalization which the author makes can be found easily. Where the attitude of the rocks is favorable, as, for instance, where they are tilted toward the east, the eastern side of a valley is usually steep and the western side gentle; but the reverse is equally true where the rocks dip in the opposite direction.

A knowledge of the geology of the country will often be of value, and the trained geologist can do reconnoissance work of this nature better than the engineer, which is one reason why I believe a railroad engineer's training should include not a little geology.

E. H. Beckler, M. Am. Soc. C. E. (by letter).—Mr. Lynch's paper treats principally of the "survey," with but slight reference to the principles of "railroad location."

I can hardly agree with him that general principles cannot be laid down by means of "fixed or inflexible rules," which should govern "location" in most all situations. Without rules, for instance, fixing a value upon distance, degree, rise and fall, radius and rate of gradient, by which comparison of alternate lines can be made, or by which a line can be adjusted to a proper expenditure for grading, the locating engineer would be groping in the dark.

On a line of road where several locating engineers were employed

there would be as many different ideas as there were individuals, and the chief engineer would find that some of his men were locating a road suitable for a trunk-line business, while others would be setting stakes for a branch road, with tri-weekly trains. There must be rules for guidance in order to produce a harmonious result.

The locating engineer should have an approximate idea of the cost of all the elements which enter into the completed roadbed, such as prices for grading, bridging, masonry, etc. In fact, he should make an estimate of the cost of each mile of road as fast as located, placing the quantities upon the profile. In this way attention will sometimes be called to unusual or expensive work, which might otherwise escape notice for the time being.

The outfit used by Mr. Lynch for reconnaissance I have found to be sufficient. The mode of transportation should be varied according to needs.

By reconnaissance made between Fort Assinaboine, Mont., and Spokane, Wash., the distance was estimated to be 518 miles. When measured, it proved to be 512 miles. Certain intermediate points were more than 10 miles off. The route was examined by four men, on different sections, with transportation by buckboard, horseback or afoot, with pack on the back, in dense forest, where five miles a day was good progress.

The manner of conducting surveys differs materially according to the character of the country. Camp equipment, kind of supplies for subsistence, mode of transportation and number of men in the party vary as the physical features and nature of the work demand.

Profiles should be made in duplicate, and the map, made on manilla paper, should be traced in the field office. Office work should be kept up with progress of field work. Maps and profiles should be in sections of about 20 miles, and forwarded to the chief engineer with all notes as fast as completed. The scales of maps proposed by Mr. Lynch I have found to be most convenient.

The daily progress of a party is a variable quantity; 1 000 ft. in some places are attended with more work than 10 miles in other localities. The only correct location for any country, requiring cuts and fills, is one made by an engineer in the field. The office projection is a help to more quickly reach the result.

I have found that equating grade at the rate of 0.04 per degree for curvature was not sufficient where the maximum grade was less than 1 per cent.

On grades of 2% or steeper there seems to be little need of transition curves. It is more convenient to have the easement curve run in, when retracing the line, as such work delays the location party. I would provide tangent room between reversing curves for this purpose. The tangent should not be less than 20 times the sum of the rates

(degrees) of the two curves. When this would make a tangent less than 200 ft., the rule should be discarded and that limit used. On cheap branch lines this tangent can be reduced to 100 ft.

I am afraid that the example cited by Mr. Lynch for a vertical curve on a summit would give a curve very much too short for such a situation. This rule will work very well where there is not too great a change in gradient. There are frequently natural obstacles which cannot be removed, controlling the curvature and gradient. The amount to be expended in changing the face of Nature depends largely upon the proposed traffic.

Mr. Henry Gannett * (by letter).—I have read with much interest the abstract of Mr. Lynch's paper upon "Railroad Location." What I have to say in the way of discussion is suggested by the last paragraph, which is as follows:

"It is quite apparent that methods such as have been described are inapplicable to countries where topographical surveys already exist, upon which the location can be intelligently projected and the line approximately laid out in the office. This system applies more for countries like America, Russia and others of large extent, where large areas are yet uncovered by topographical surveys."

The advantages of such topographical maps to the location engineer are apparent. They obviate the necessity of making any preliminary reconnoissances, since from them all feasible routes which are worthy of being examined in greater detail can be selected at once. Not only this, but in most cases an examination of a map reduces the routes which it seems worth while to examine more closely to one, two or three, and thus obviates the necessity of running many trial lines. Moreover, when a route has been thus selected, the engineer has the fullest possible assurance that he has the best route which the country affords. It is impossible that further exploration of the country can develop any better route than that already selected. With so many cases before us of badly located roads, roads which have been located badly because of an insufficient knowledge of the country, roads which have been shifted and straightened after completion, because a more intimate knowledge of the ground has shown that better routes existed, it seems strange that different methods of making preliminary reconnoissances have not been adopted.

Now, what I wish to advocate is the substitution for this preliminary reconnoissance work, which is treated of in this paper, the making of a topographic map. Such a map should embrace a belt of country between the points which the railroad must touch, of a width sufficiently great to include within it all possible routes. For instance, in the case which Mr. Lynch cites of a road between terminal points 250 miles apart, touching in its course several intervening towns, it is

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probable that a map of a strip of country 25 miles in width will cover all possible routes. Such a map of an area of 6 250 sq. miles, which will answer the requirements of preliminary railroad location, can be made at a cost ranging from $60 000 in the eastern part of the country down to $10 000 or $15 000 in the Mississippi Valley and on the plains, and up to perhaps double that amount in the Rocky Mountain region. I do not know what relation this expense would bear to that usually incurred for reconnoissance surveys, which it is designed to supplant; but I think I am safe in saying that the latter would cost several times this amount, besides, as has been shown above, meeting the requirements in several respects very inadequately. I therefore suggest for the consideration of engineers engaged in railroad location the advisability of making this radical change in methods.

J. Q. Barlow, M. Am. Soc. C. E. (by letter).—I believe the points in Mr. Lynch's paper are generally well taken.

After an experience embracing many thousand miles of reconnoissance and surveys in the Rocky Mountain and Pacific Coast regions, the writer believes the correct balance of all the conditions involved is often the hardest of attainment in what, at first sight, appears to be a region of no special difficulty. Where the position and direction of the drainage is favorable between objective points, but little difficulty is experienced in deciding upon the route. In rugged and mountainous regions involving carrying the line across one or more summits, the problem is one of more difficulty, but a thorough reconnoissance generally brings out the distinctive advantage of some particular line as one of unquestioned superiority.

But most experienced engineers have at some time or other been confronted with a wide area of country over which it was necessary to locate a line, and while in no sense mountainous or even rugged in appearance, yet upon detailed examination proved to be full of gulches and swales, somewhat rolling and having no well-defined system of drainage in any given direction as a whole, and somewhat similar, probably, to what Mr. Wellington in his book describes as "a country worse than it looks." It is such a country as this which may, at a distance, have the innocent appearance of a gently rolling plain, but which, upon closer examination, often tries the patience, wits and muscular endurance of the engineer, in order to formulate satisfactorily unto himself the best plan of location across a region apparently all warped out of shape.

The writer has often noticed the point brought out by Mr. Lynch with regard to the north slope of east and west valleys and gulches being much less rugged, and has always thought it to be due to the fact of greater exposure to the sun in winter and consequent freezing and thawing, which for ages have operated to disintegrate, wash down and smooth over to some extent their former roughness; and this

fact often cuts a considerable figure in carrying a supported line off from a summit, as well as more freedom from snow in operation.

Mr. Lynch recommends an 800-ft. scale for preliminary maps, and in difficult sections a 400-ft. scale. In all grade line and contour projection work this is entirely too small; in difficult mountain and cañon work 100 ft. to 1 in., with 10-ft. contours, gives best results. On all but the most difficult work a 200-ft. scale gives satisfaction and admits of projecting with such accuracy that a very good profile may be made from the map and but few corrections will have to be made in the field when the line is run in on the ground.

No standard can be set as to the amount of work accomplished in any given time by a field party. A party under the direction of the writer, while crossing a portion of the Great American Desert, in Western Utah, where no water exists for a distance of about 100 miles, nearly exhausted the water supply carried when about 70 of the 100 miles had been run. The following morning at 5 o'clock a start was made with a view to running 30 miles by dark and reach water. By quarter of two o'clock 20 miles had been run, including the levels, being 8 hours and 45 minutes of actual time, which includes about 15 minutes for lunch, and is, the writer believes, the best record for the time, and perhaps for a day, ever made. Owing to the teams giving out at this juncture, further progress was for the day abandoned.

The levelman used two rodmen, and a 200-ft. chain was used; but the most extraordinary part of the performance is perhaps the speed of the leveler, but it is accounted for by a clear atmosphere, very level country, and rapid manipulation. It might further be mentioned as a matter of interest, that this work was a portion of a circuit of about 1 700 miles of levels which closed within 7 ft.

There is a wide difference from this extreme to that of difficult mountain work, where sometimes for weeks together an average per day of less than ¼ mile is accomplished; and in one instance, where one of the writer's parties was nearly six weeks in locating 1½ miles of extremely difficult cañon work, including the same length of preliminary line, much of the ground on which the line was located being inaccessible, except for a bird, a large amount of complicated triangulation was necessary, and often from the opposite side of the cañon.

It cannot be denied that there are many examples of poor location to be met with, which are hard to justify under any conditions; but on the other hand, we sometimes hear harsh and unjust criticism on the location of lines, considering only the conditions they are being required to fulfill at present, and with no regard for the conditions as they existed in fact and prospective at the time the line was laid out.

Then, the balance of business may have been eastward; now, it may be westward ; then, five trains daily may have seemed a large estimate ;

now, 25 trains may be run daily; then, the young company may have been struggling and poor, and the fact may to-day be reflected in its location, and rather than being proper subject for criticism, it is perhaps the wonder that the earlier railway locators builded as well as they did.

Mr. W. M. Davis* (by letter).—Mr. Lynch's statement regarding the position of steeper slopes in valleys seems to me open to numberless exceptions, and therefore of problematical value in so important a matter as railroad location. The factors on which steepness of valley slopes chiefly depend are, the structure of the region traversed by the stream; the age of the valley (that is, the stage of its geographical development); the meandering of the stream; and, in a subordinate way, the action of sunshine on the slopes; the action of wind; the presumed influence of the earth's rotation. These several controls may be briefly considered.

The structure of the region is all-important. Horizontal strata allow the development of symmetrical slopes on either side of the stream; but if the stream meanders, the slope on the outer side of its curves is the steeper, as is well known. The gorge of the north branch of the Susquehanna in Pennsylvania, the valley of the Osage in Missouri, and many others, might be mentioned in illustration of this fact. Where the river course is comparatively straight, the valley slopes are relatively symmetrical; but where the river course is tortuous, the banks are often extremely unsymmetrical; and the steeper slope is then on the outside of the curve, whether this be east or west, north or south.

Tilted stratified structures nearly always determine an unsymmetrical form of valley section; but no definite rule can be given for the steeper and gentler slopes, as they depend on variable factors, such as thickness and resistance of the several strata concerned. In monoclinal structures of moderate dip, it is, however, most commonly the case that the steeper slope is on the side of the descent of the strata. This is illustrated over and over again in the Appalachian belt.

Massive crystalline rocks are so complicated in structure that no simple statement can be made concerning the form of valleys carved in them.

The age of the valley is also a matter of great importance. Young valleys are always steep-sided; for example, the gorge of the Niagara or the cañon of the Colorado, their depth depending more on the altitude of the country than on their age. Adolescent valleys have gentler slopes, as along the Ohio. The Connecticut valley is older, and has generally wide, open meadows, as by Hartford; but where the rocks along its path are hard, the slopes are still steep, as below Middletown.

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The effect of a meandering course has been mentioned above. This effect is locally strong enough to form exceptions to the rules based on the other kinds of controls. Illustrations of it may be cited in great number.

The smaller controls—sunshine, winds, and the earth's rotation—are comparatively problematical. I believe that engineers find that the sunny slope of a railroad cut is the more readily attacked by the forces by which rocks are decomposed, presumably because of the greater number of strong temperature variations to which it is exposed. The same principle might apply in forming large valley slopes, provided other controls were indifferent. If sunshine were the determining cause, a slope to the south-southwest (in our temperate zone) might be more generally of moderate declivity than one in the opposite direction; but I think it must be rare that this control acts alone, or as a determining cause of form.

The action of the wind may be mentioned; but it is probably negligible in small streams, and other controls may overcome it, even in large rivers.

The effect of the earth's rotation has been much discussed, and both sides are ably argued. It should be noticed that this effect, if apparent at all, should be seen on the right bank of rivers in this hemisphere, and on the left bank in the southern hemisphere, while its intensity should increase with the sine of the latitude. The statement ordinarily met with, that the deflective effect of the earth's rotation appears only in the meridional component of a river's motion, is incorrect. The deflective force is independent of the azimuth. A river running east or west is deflected just as much as a river of the same volume and velocity, in the same latitude, running north or south.

But, at most, this action must be small. Other controls must generally exceed it, for it is by no means a general rule that the right slope of our valleys is steeper than the left slope.

The valleys of certain streams in North Carolina and in Long Island have been instanced as exhibiting a form that might be ascribed to this control, as no other control is there apparent for their unsymmetrical form. The best examples of the kind that I know of are along the radial streams of southwestern France, excellently shown on the Army Staff map of that country (1 : 80 000); the streams run in nearly all courses between east-northeast and west-northwest, and nearly every stream has a steep slope from the hills on its right.

Now, Mr. Lynch states that "where a stream flows north or south, the east slope of the valley is invariably steeper, rougher and more broken than the west slope," and "where a stream flows east or west, the south slope of the valley is generally steeper, rougher and more broken than the north slope." It is manifest that if these statements are correct, there must be some predominant control of valley form

besides those generally recognized and mentioned above; and this conclusion is so unlikely that I am constrained to question the general applicability of Mr. Lynch's theorems. It can hardly be maintained that the rule of steeper slopes on the east, in north and south valleys, is "invariable;" the Hudson at New York City and for some miles above is a contradiction to that rule; but it accords with the rule of structure, for it follows the strike of a monocline structure, whose dip is westward, and its steeper slope is therefore on the Palisade shore. Many other exceptions to the rule might be named.

As to the confirmation of the rule by the continental example of North America, it seems to me very hazardous to introduce forms so unlike in dimension as valley slopes and continental slopes, as if they belonged in the same category. It is, of course, well known that both North and South America have the greater mountain ranges near their western coasts; but Australia has its mountains near the eastern coast. I hope that in the fuller publication of Mr. Lynch's paper, he will indicate by specific statement, the observations on which his rules of valley form are based.

It seems to me that if the engineer is to take advantage of the form of valleys in locating railroads—as he certainly ought to—he must be guided, not by empirical rules, to which many exceptions will be found, but by a practical knowledge of physical geography and physical geology. There is no other subject of study that so greatly increases the power of rapid perception of topographic features; and as Mr. Lynch's paper indicates, that power is of extreme importance to an engineer at work on new railroad locations.

Kenneth Allen, M. Am. Soc. C. E. (by letter).—There has been some discussion of late years as to the merits of the stadia for preliminary surveys. I think that depends entirely on what is required of the survey and on the character of the country traversed; but where the latter is open, and the feasibility of a project, the ruling grades and an approximate location of the best route are acquired, I think the stadia method will be found of greatest value. The saving in time and expense will be found very considerable and the above information with as minute detail of topography as is wished may be obtained, the latter work being extended *ad libitum* according to the ground covered. In other words, a much larger fund of general information may be obtained for the same money than by the usual method.

A survey of this kind properly belongs in an intermediate place between the reconnaissance and the chained preliminary survey, and under certain conditions it may prove an economical substitute for both. The economy, of course, results from the absence of chaining and stake driving and the consequent reduction in the size of the party. The line, however, should be so established by referencing turning points or angles that it can be recovered readily.

I do not consider the error in distance or level to be so much greater by this method (if, indeed, it is greater in the long run) as to be a serious objection to it. And although it may be easier to err in reading the stadia rod than to drop a station in chaining, the transit-man will probably guard as much against the former as the chainman against the latter.

Having had an opportunity some six years ago to test this method of making a preliminary survey where the object sought was: 1, to ascertain the feasibility of constructing the road; 2, to establish the best general route, with distance, elevations and ruling grades; and 3, to furnish an approximate estimate of the cost of construction, I adopted it, and the following notes with reference thereto may be of interest:

The line ran almost entirely over a prairie country in Kansas and Nebraska, considerable of it cutting "across country" and requiring numerous trial lines to be run for comparison.

The established line was 312.13 miles in length, and the time taken for one party * for the entire field work was 5.7 months; or 2.1 miles of line were established per day with a party of transit-man, leveler, two rodmen, axeman and teamster. Deducting rainy days and time consumed in moving, the progress would probably have been about three miles per day. † The cost for the entire field work for salaries and expenses, but not including instruments, camp outfit and office work, averaged $11.03 per mile.

The work consisted in running bench levels, the projected line by stadia, including elevations of all controlling points on or near the line with vertical arc, referencing turning points and locating township and county lines. In addition, 23 miles of alternate line were run and a very considerable extent of territory reconnoitered on horseback.

The saving arising from the use of the stadia on this survey was probably at least 25 per cent.

Where one is familiar with the territory beforehand, where good contour maps of it exist, or, in other words, where the final location will necessarily be confined between comparatively narrow limits, and also in a wooded country—and these conditions obtain in a large number of cases—I am inclined to favor the ordinary method with transit and chain. Mr. T. G. Gribble, however, has employed the stadia with success, as he claims, not only for such preliminary work, but in the actual location, including the staking out of curves, of 70 miles of railway in Hawaii. For this, however, he employed a special instrument, magnifying 40 diameters instead of 25 or 28, as customary.

On a preliminary survey for a conduit route made about eight years

*Two parties were in the field a part of the time, but such time is counted double.

† Eight miles were run on one day. An axeman was not required on the entire survey.

ago for the city of Philadelphia by Rudolph Hering, M. Am. Soc. C. E., the following method was used with satisfactory results, the alignment and level checks indicating accurate results.

Bench levels were run near the projected line with double turning points. They were run in this way in order to avoid duplicating the work for a check, although a check was obtained subsequently by the completion of a circuit.

The transit party followed, driving stakes at turning points and at stations 200 ft. apart. Two transits were used, occupying alternate turning points, sighting to each other, turning vernier angles and checking with the needle. Elevations were carried by both transitmen with the vertical arc, checking on level benches. At each point occupied topography was taken with stadia and level rod for some distance each side of the line, depending on the ground, and continued by the rodmen by use of an Abney level or Linton's clinometer.

The results were platted on a scale of 800 ft. to 1 in. with 5-ft. contours, and on this plan the final location was projected.

The data as to cost and rate of progress I am unable to give, but my impression is that the latter was something over one mile per day on average ground, and that the party consisted of two transitmen, one leveler, three or four rodmen and one axeman.

With regard to such surveys, I would suggest the following : Run a line of bench levels ahead of the transit party, establish bench marks every half mile or so for the transit party to check on. Select a transit of good magnifying power and a stadia rod carrying a distinct graduation. Black and white has been found far preferable to other combinations for the entire graduation. Start on the true azimuth and turn angles on the true azimuth at each set-up, always reading the needle as a check. In more rapid and less precise work the needle alone may be used and alternate turning points only occupied by the instrument. On extended surveys check the bearings by an occasional observation for the meridian. Observe the position and elevation of all points controlling the location. Carry elevations through by the vertical arc.

Do not delay the work by attempting to keep the traverse coincident with the final center line where obstacles occur, as this line can be projected on the map and scaled or calculated, and will, moreover, as often as not, vary from the line as finally established.

Foster Crowell, M. Am. Soc. C. E. (by letter).—This paper, as its author intimates, aims to be suggestive rather than instructive; it opens a very wide field for discussion without much specific reference. Nevertheless, it may prove very useful by bringing out and emphasizing the undeveloped condition of the art of railway location, which is a marked and surprising feature in the greatest country of railroad builders in the world. If we compare the field methods of to-day with those of 40

years ago we will find there has been very little advance in the art, however much theory may have become enlightened, and despite the fact that in other surveying fields, such as park-making and suburban street development,. for example, there have been marked improvements in instruments, methods and results. This is, doubtless, not entirely the fault of the profession ; part of it is due to the ignorance and false economy of employers. There are, of course, some exceptions to this general statement ; the aneroid stadia, plane table, and the camera have, sometimes, perhaps, and to a small extent, been employed for railroad surveys, and sometimes sufficient time is taken to thoroughly study the country before beginning the location. But, nevertheless, it is true, as a rule, so far as I have had opportunity to observe, that the old way is adhered to, of first of all putting a party into the field with a chain and transit, followed, instead of preceded, by a level, and blundering along as nearly as possible in the desired direction on a line that is considered to be tentative, but yet is expected to conform closely and to be a basis for the location to follow. The programme of preliminary, preliminary location, and final location, is in view, in a more or less complete form, but, as usually practiced, the first is commonly determinative of the last, and the test of a good locating engineer in the minds of laymen is as to how close he can adhere to it, or, in other words, how narrow can be the belt of examination. It is not to be denied that excellent results are often thus obtained by skilled engineers; but, on the other hand, this method does not, in any case, demonstrate that the best result has been attained.

Another test that is often applied of the proficiency of a locating engineer is his linear progress per day, a party who can cover two miles per day in a moderately difficult country being perhaps considered a hustler, while the party who makes but half a mile per day is voted slow; whereas, the results of the careful progress may save in the construction account many times the extra cost of the survey, for it requires but a small percentage of distance saved to meet the latter, not to speak of other obvious advantages to be gained. And such locations are naturally and usually very defective, especially in the details, some of which, such as "broken-backs" and reversions, Mr. Lynch has mentioned. The evils of "crowding the engineer" in the field are not overstated by Mr. Lynch. It frequently happens, too, that branches or extensions reflect and repeat errors with which the management may be already familiar on the present line, and the standard is, therefore, seldom ideal. Perhaps this accounts, in part, for the impression that appears to prevail in many high quarters that location is a comparatively unimportant thing and easy of accomplishment at sight.

Sometimes engineers are sent into the field with imperative limita-

tions (which are apt, in such cases, to be far too high) of grade and curvature, and instead of considering them as limitations, and only to be resorted to when unavoidable, they sail serenely along, using the maximum all along *ad libitum*, often to save light graduation costing less per linear foot than the extra length of track, and throttling the traffic capacity of the future road to a still more injurious extent. For all of these obvious reasons all the preliminary topographical information that can be obtained and utilized prior to the beginning of the line survey, within common-sense limits as to width of territory to be examined, should be fully studied and detail surveys and studies with stadia, camera and other labor-saving devices, should be made of such localities as present obvious alternative locations and the more difficult ground. This course will, in most cases, cost less, and generally be far more valuable than a complete continuing preliminary line survey, even if it does not further admit of a final location being made from it without preliminary location. There are, also, obvious advantages from the right-of-way point of view, in knowing the location before running on the actual line, and the cost and delay of revising the tentative line at difficult points is largely avoided.

Mr. Lynch's remark that railroads cannot properly be definitely located from a preliminary map in the office, and that the best results must be actually executed on the ground, is true, but only in a measure. It all depends on the preliminary map, and the method of its production; he probably wishes us to read between the lines. Branches of trunk lines may, sometimes, it is true, be located in an arbitrary and ill-considered manner from the general office.

I return for a moment to the author's observations of the greater steepness and roughness of the south slopes and the east slopes of valleys in this country. I do not know that I quite understand whether he refers to the declivity of the valley itself or of its cross-section; if he means the latter, I cannot say that my recollections bear out the statement sufficiently to make it general; not only are there marked departures from the rule in the case of separate streams, but the conditions vary in the same streams, the steepness being sometimes on one side and sometimes on the other. Whether true or not, it has not much bearing on location problems, however interesting geologically or geographically.

Altogether, I think the profession is indebted to Mr. Lynch for the suggestions in his paper, and, although, to some, they may be more or less self-evident, still, in view of the way they are generally disregarded in actual practice, they should be welcomed.

Charles H. Snow, Assoc. M. Am. Soc. C. E. (by letter).—I have repeatedly been engaged in exploration work in the wilder portions of this continent, where horses could in no part of the work be used. In such instances, if parties are to be detached from sources of supply

for considerable intervals, the selection, as well as the transportation, of provision, shelter and instruments, will require the most careful consideration. This is a subject which has frequently not received the attention which it deserves, yet the character and quantity of work accomplished is directly affected by it. I have met more than one engineer who has been the victim of the ignorance or carelessness of others, even to incipient scurvy.

In my own experience, when all were upon short allowance and no supplies within five days' march, I had once to abandon provision which was perfectly good, but too bulky and heavy to get across an extended wooded swamp.

Food should be selected with regard to climate and the habits of the men employed. It should be nutritious, in variety, easily and quickly cooked, and compact. Canned vegetables and fruits are to be avoided as containing too much water, and, therefore, heavier, while no better than the same in desiccated state. Supplies are generally packed in small canvas bags, which are baled by wrapping them in the blankets, tents, etc.

Pack-straps, easily adjusted to any package and shoulder, are everywhere procurable. Packmen should hardly be loaded with more than 50 or 60 lbs. each, although I have known exceptional men to handle over 100 lbs. in rough country and for prolonged distances.

Extra compasses should always be carried. Long wire nails for improvising rafts for unexpected lakes are frequently invaluable. In explorations, as in other kinds of work, better results can sometimes be obtained in winter than in summer. Swamps are then frozen, and dog teams can frequently be employed, even in thick brush or over wind-falls. Toboggans drawn by dogs were used on parts of the Duluth, South Shore and Atlantic Railroad during early construction, the partially cleared location transit line furnishing fair going for the dogs. Topographical work can be done in wooded country as well in the winter as the summer, as the vegetation forms a satisfactory guide to the character of the ground. In warm climates, as on the Mexican table lands, outdoor work is to many more agreeable during the rainy season than in the hot dusts of the dryer portion of the year.

Foot-gear is another direct factor in the comfort and health of men upon the march. This is a point of sufficient importance to have received attention from the German and perhaps from other governments. Where in ignorance of conditions to be met, a serious mistake can hardly be made in selecting a shoe or boot pack. This, as known, is a sort of moccasin with heavy sole. It is manufactured throughout the east and west. For winter work, in cold latitudes, a thick felt stocking, belted above the knee with light rubber sandal, is widely used. In temperature as low as — 40° Fahr., and in deep snow, I have found this arrangement to be warm, comfortable and dry. Rubber boots are very unsatisfactory in all work of this description.

In reconnaissance through dense forests, the position of the party may be kept by a leader furnished with a large hand compass, with needle not sensitive, but which will quickly come to rest. The chief, particularly if unacquainted with the diverting hardships in unsettled districts, should not attempt to do this work. After a little practice the results obtained in getting round swamps, in making detours to cross rivers and yet come back again to the line of travel, is quite as surprising as the measurements determined by timing well-gaited horses, as mentioned by Mr. Lynch. Courses and distances are kept by the leader with his compass and by pacing. It is easily possible to travel two or three days away from camp in brush so thick as to obscure objects 100 ft. distant and over country without the slightest individuality, and returning another way, to strike camp closely. The quality of location work might sometimes be improved, if the construction engineers were placed upon the ground sufficiently early to study their separate divisions from a location standpoint. Sometimes slight improvements, in no wise involving the general system of location, can be introduced at large saving.

J. E. Greiner, M. Am. Soc. C. E. (by letter).—The author has paid little attention to one feature in location which, though probably of minor importance compared with the proper selection of route, grades and curves, still forms a part of the location of every railroad. I refer to the choice of bridge sites and the selection of the proper design for any particular opening.

Bridging of small streams is not usually done in the best and most economical manner. Spans are made too short and water-way is blocked, made too low and damaged by freshets, made too long and money wasted. Each locating engineer considers himself perfectly competent to fix upon the length of span and style of structure required for any particular stream upon a casual inspection of the crossing, although it has been demonstrated over and over again that the selection of a bridge, even for the most innocent-looking stream, requires careful judgment, based upon an accurate knowledge of the nature of the stream, its water-shed, its liability to a sudden rise and the height of freshets. A poor selection is sure to be exposed, sooner or later, to the maintenance officials after the road is in operation, while the locating engineer may be making the same selection elsewhere in total ignorance of previous poor judgment.

There are a few recommendations, mostly of a negative character, which may be worth considering in this connection.

First.—Limit the skew to 45°. For single crossing it is better to avoid skews. The stream, of course, can be crossed at any angle, but make abutments square with the axis of the bridge, or as near square as circumstances will permit, and the saving in masonry will frequently balance the increased cost of a square bridge with longer span. Bridges

with sharp skews cannot always be braced in a satisfactory and thoroughly efficient manner; besides this, the load coming upon one truss before it reaches the other is apt to cause racking or distortion to a more or less injurious degree.

Second.—Avoid sharp or broken-back curves. Bridges with curved alignments cost more, are more difficult to maintain and show the effects of wear in a shorter time than when straight. Knowing this, some engineers make a short tangent over the bridge, the P. C. being very near each abutment. This arrangement is just as hard on the bridge as is a curve, because the train has to recover from the effects of the centrifugal force caused by the curve at one approach, and then, almost before equilibrium is restored, prepare itself for the curve at the other approach. All this disturbed equilibrium takes place on the bridge and is not always considered in the design. The data furnished stating merely that alignment is straight, the designer, unless a bridge engineer in the employ of the railroad, does not go back of this information, and designs the bridge accordingly. The result is a weakness which soon makes itself felt.

Third.—Avoid half-through girders for small spans. The relative cost of deck and through-plate girders designed for 125-ton engines, erected ready for the rails, is given approximately in the following table, as is also the least distance from base of rail to under clearance:

Span.	Deck.			Through.		
	B. of R. to under clearance.		Cost.	B. of R. to under clearance.		Cost.
Feet.	Feet.	Inches.		Feet.	Inches.	
30.....	3	3	$525 00	2	3	$975 00
40	4	1	800 00	2	3	1 400 00
50.....	5	0	1 160 00	2	3	1 910 00
60.....	5	11	1 610 00	2	3	2 510 00
70.....	6	8	2 120 00	2	4	3 170 00
80.....	7	6	2 700 00	2	4	3 900 00
90.....	8	4	3 375 00	2	6	4 725 00
100.....	9	2	4 125 00	2	6	5 625 00

An examination of the above table will show that through-plate girders cost from 36 to 85% more than deck; that for the same money a deck girder can be built about 17 ft. longer than a through; that with 12 ft. depth of water a deck bridge costing the same as a through bridge can be made to have the same clearance as through girders up to 50-ft. spans, and with 20 ft. depth of water up to 70-ft. spans.

There is, of course, no objection to the use of through-plate girders when circumstances so demand, but the deck girders giving a roadway clear of all obstructions should always have the preference.

Single-track through-plate girders over 8 ft. deep, having a very shallow floor, as, for instance, when cross-ties rest directly on bottom

flanges, are abortions. The cross-struts and knee braces do not stiffen the top flanges nor hold them in line; when a train crosses, the girders cant inward and convey the idea that they are going to upset. The rushing of trains over such structures makes a thundering noise very objectionable when heard unawares or at night.

Fourth.—Avoid using three trusses or girders in double-track structures. There are cases when such a design will save material, and consequently cost less than by using two or four trusses or girders, but an examination into the action of such structures in actual service has convinced the writer that they are fallacious in spite of all care taken to proportion the trusses and bracing for existing conditions. The trusses will not work together except when both tracks are loaded. When the train is on one track only, the unequal deflection will distort the bridge and loosen rivets in struts and floor beams, while in deck trusses the sway frames cannot be kept in adjustment, or when made of shape iron and riveted, the rivets pull loose. In cases of adjustable sway rods, the continual tinkering to which they are subjected will eventually so distort the cross-section as to cause the bars in the various panels to take unequal strain. The writer knows of a bridge of this description having three-deck trusses resting on artificial stone pedestals. The bed-plates in the outer trusses have worked into the stone about 1 in. during a period of about nine years, while the bearings under the center truss have not worn more than $\frac{1}{8}$ in. The distortion in the case therefore is about $\frac{7}{8}$ in., and something had to give this amount. The cause is evidently due to the unequal straining of the outer and center trusses when one track is loaded, the outer truss taking its full load, while the center gets but one-half its load; consequently, as two trains seldom meet on the bridge, outer trusses do about twice the work done by the center truss, and the soft-stone bearing under the outer trusses will naturally wear much faster than the center bearings.

Attempts have been made to formulate rules for the most economical layout of spans over a wide stream or river.

One author states that this is effected when the cost of trusses and lateral system about equals the cost of the substructure, and gives a mathematical proof of the statement. Whether true or not, such a rule is of no practical value. A rule cannot be made to fit all cases, and the best plan therefore is to find the best layout by trial estimates. That these estimates should be made by the bridge and not the locating engineer goes without saying, or if the company has no bridge engineer, they had better submit a cross-section of the stream showing the nature of soil and the established grades to a reliable bridge company and allow it to determine the layout.

James R. Maxwell, M. Am. Soc. C. E. (by letter).—The paper of Mr. Lynch is admirable. As he says, "no fixed or inflexible rules for location can be laid down." Each case presents some new problems that

must be solved on the ground. The maps of the States and territories where the land has been subdivided by the Government are more accurate than those in the other States. With these maps and a good aneroid barometer, a very good reconnoissance can be made, and the position of the line and the maximum grades closely determined, if the weather is settled. It is better to have an assistant, and two or more barometers along, so that in unsettled weather one observer can be left at a controlling point, either the summit or the valley, to note all changes and the time of their occurrence; the other, to go ahead and take his reading and the time; then return and check up, and if there is any material difference, to repeat the readings, but exchanging the barometers. This causes much work and delay, but it pays to make sure.

My experience in the States of Washington, Nevada and California agrees in general with what Mr. Lynch says about the steepest slopes and the roughest ground being found on the west or south side of the ranges. A very marked instance of this was found in the southern part of California, in crossing the Sierra Nevada. On the east side, light work, with 1% grade, and but little curvature was encountered; while on the west side, there was very heavy work, a large amount of maximum curvature and 2% grades. In the southern part of Pennsylvania, the light grades are found on the western slope of the Alleghenies. The locating party should always be provided with a team to carry them to and from work, and, if the survey is of any magnitude, with a full camp outfit also.

As Mr. Lynch says truly: "There is no royal road to railroad location; it must be executed on the ground," and permit me to add that it should never be considered finished until the grading has so far advanced that any more changes are impracticable.

Harvey Linton, M. Am. Soc. C. E. (by letter).—A piedometer adjusted to show correctly the distance traversed is a useful addition to the instruments carried in a reconnoissance.

As the location of a railroad in countries where topographical surveys exist may be satisfactorily determined in the office, it seems reasonable that some of the methods used in making these topographical surveys may be applied with advantage in preliminary surveys for railroad location, particularly in open and rolling country.

The methods of stadia surveying for a railroad line may vary in details from those used in filling in large areas of topography. In such a survey each station or turning point on the line should be occupied by a theodolite or a transit instrument (the transit feature of the instrument is not required), so that the azimuth angles may be measured.

If there are two instruments and two surveyors, the distance and the difference in elevation, or the vertical angle between any two sta-

PLATE VIII.
TRANS. AM. SOC. CIV. ENGRS.
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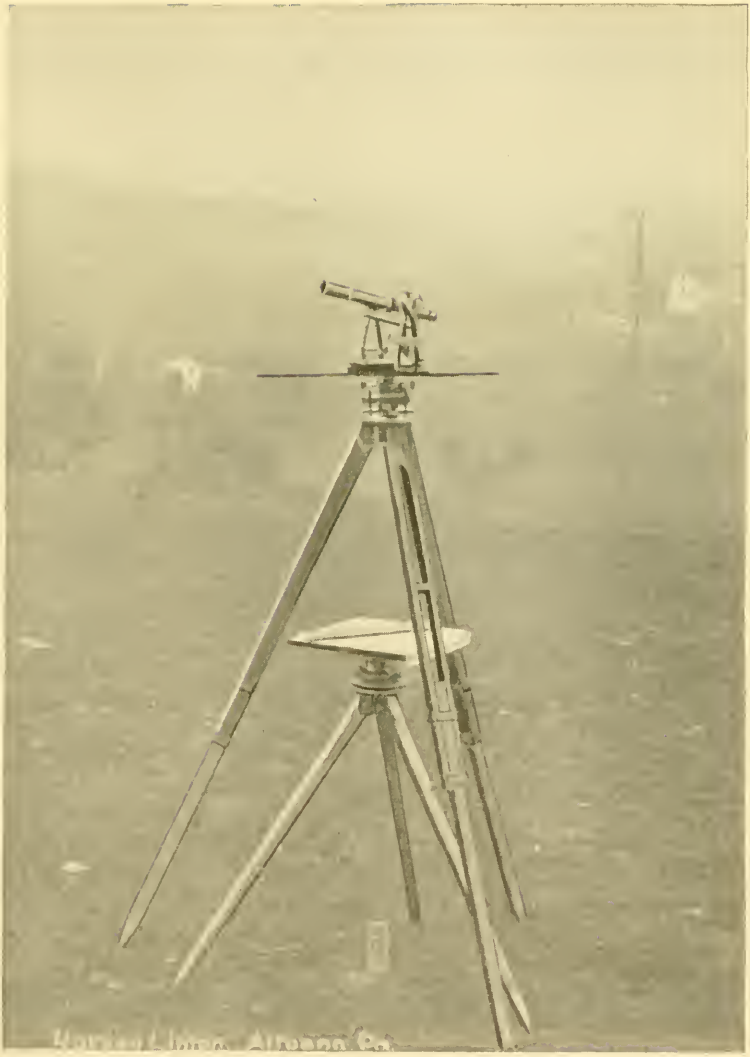
LINTON ON RAILROAD LOCATION.



tions, may be read from both of these stations simultaneously, as a safeguard against any serious errors of observation.

The topography can be plotted in the field in fair weather upon small plane-tables oriented at each station (Plate VIII).

A party of six—two instrument-men, who are also topographers, and four rodmen, each provided with a rod and a light axe—will do much more work in the same time than a larger party, with chain and stakes, and instead of notes they will send into the office sheets of topography, the only field-notes necessary being those relating to the length and bearing of the lines of the survey and the elevations of the stations or turning points.*

Mr. Bailey Willis (by letter).—The location of a railroad is controlled by the topography of the region through which it passes, and topography is related to geology; therefore, as a geologist, I may venture to consider what Mr. Lynch says of topography.

Mr. Lynch rightly says that an engineer should be thoroughly familiar with the drainage of the district, and that he may infer the direction of steeper slopes from the relative nearness of a main stream to a divide. But the broad generalization that the south slope of an east and west valley and the east slope of a north and south valley are steeper than their opposites, is, I think, a conclusion which must admit of as many exceptions as there are instances to support it.

Consider what determines slope. The first condition is difference of elevation and nearness of summit to lowland. But if we eliminate this primary condition and consider slope in two directions from a given summit, we shall recognize that the steeper slope is always across the dip of the rocks, the gentler slope is always with the dip of the beds.

Relief is carved from the rocks by water; soft rocks, yielding, make valleys; hard rocks, resisting, make elevations. If the rocks occur as strata which are alternately hard and soft, the form of relief depends upon the attitude or dip of the beds.

First.—If the strata extend horizontally, the channel carved by any stream is a cañon, having vertical walls in hard rocks, supported by talus slopes in the underlying softer strata. The cañon of the Colorado is the type: examples are common in the Cumberland plateau of Kentucky, and the "table-mountain" or "mesa" is the characteristic hill.

Second.—If the strata are inclined, the work of erosion produces a more or less pronounced cliff-and-talus profile across the dip, and a long gentle slope with the dip. The ridges of the Appalachian belt are the type; examples are common along the foot-hills of the Rockies, and the characteristic form of relief is the long, even-topped ridge.

*Proceedings of Engineers' Club of Philadelphia, Vol. VIII, No. 2, Topographical Surveying.

Third.—If the strata be vertical, the work of erosion is usually equal on both sides and the ridge is symmetrical. Examples of this occur in the Appalachian belt, and in other zones of upturned rocks.

The subject of topographic form is a very broad one, and I submit these brief comments merely to call attention to the fact that slope is not a function of the point of the compass, as Mr. Lynch seems to imply.

In regard to instruments I note that Mr. Lynch does not mention the plane-table. This instrument is not in favor among railroad engineers, but its development and application in the last five years in the United States Geological Survey may justify the statement that it will be found most useful to them, and particularly in making locations.

Charles W. Staniford, M. Am. Soc. C. E. (by letter).—During the location and construction of the South Pennsylvania Railroad, from 1881 to 1885, all grades were compensated on curves at the rate of .02 per station per degree of curve, and changes in grade were effected by true parabolic curves, usually about 600 ft. long.

Further than this, after the final location had been adopted and the work let for construction, a degree of exactness in this direction was ordered which I do not think has been attempted on the original construction of many other roads, as follows: On a grade line where the curve compensation makes a lightened gradient for the length of each curve, forming at the beginning and end of same a break in the grade, parabolic curves were introduced at each of these breaks 300 ft. long, running 150 ft. in each direction from the P. C. and P. T. of the curve. These easements eliminate all angularities and transform the entire grade of the road into a series of tangents and parabolic curves.

It would appear that, although the corrections necessary to round these slight breaks in a gradient are usually small, the result from their use is part of the perfection arrived at in the maintenance and improvement of many trunk lines, and that some extra work would therefore be warranted in the calculations for the grades of such roads during the first construction of the roadbed.

As a matter of fact, however, after the elevations had been corrected for these curves, at the beginning of the work, we had no more trouble or labor during construction in using gradients so treated than in similar work on regular grade lines.

P. F. Brendlinger, M. Am. Soc. C. E.—The paper before the Society this evening is exceedingly interesting, and should be discussed and entered into with zeal by all the railroad engineers present, for the occupation of "Railroad Location" was, and still is, to a great extent, the idea of the people at large the definition for the business of a civil engineer. The badge we carry is likewise emblematical of the same.

The preparation of the paper shows Mr. Lynch to have covered the subject quite clearly, in the main, and, of course, it would be very

strange if in the matter of details there would be no differences of opinion.

In regard to finding the steepest slopes on the south side of mountain ranges extending east and west, and on the east side when a stream flows north and south, I cannot say that the rule is general, but it seems that the majority of railroads now constructed and in operation do have their maximum grades on the south and east. The Pennsylvania Railroad, the Baltimore and Ohio Railroad, and all roads south of them crossing the Allegheny Mountain Range, to the best of my knowledge, have their steepest grades on the south and east sides. The uncompleted South Pennsylvania Railroad is similarly situated. The Lehigh Valley and the Central Railroad of New Jersey in crossing the Blue Ridge are an exception to this rule. The rule seems to hold good, also, in crossing the Rocky Mountains, with the exception of the Great Northern Railroad.

Chief Engineer Beckler, in his written discussion to-night, does not mention or discuss this part of Mr. Lynch's paper, yet in an article in a late issue of the *Railroad Gazette*, written by Mr. Beckler, he gives the profile of the Great Northern Railroad, which shows that the steepest grades are on the western slopes on the Rocky Mountains.

In my own experience I had never discovered this supposed truism; in fact, it is something new to me, never having seen it in print nor heard of it before.

Reconnaissance.—Mr. Lynch recommends using a gaited horse for making reconnaissances. I think this a rather loose and unsatisfactory method, and, as chief engineer of a road, would hardly pursue it. If a country, through which a reconnaissance is desired, is rough and sparse of good roads, I would have three saddle horses for the party, led during working hours by one man, while the other two men made the reconnaissance with a stadia instrument on foot, or with aneroid and pedometer. At close of the day the horses could be used to find shelter for the night. I have always used the aneroid for reconnaissances and have made some remarkably close observations, frequently having found the difference not more than 4 or 5 ft. from the actual levels in a distance of 10 miles. Mr. Lynch mentions that "settled" weather is necessary for good aneroid work—I presume he means clear weather. I have made just as close observations in cloudy weather as in clear, that is, not partly cloudy and clear, but when the sun was obscured all day.

Camping.—I think Mr. Lynch is rather severe in his censure of stopping at farmhouses while making a survey; he calls it a "one-horse arrangement." Camping out in tents is quite romantic, convenient and sure, but who can estimate or name the number of professional railroad engineers who are not afflicted with rheumatism contracted by camping out? I know, to my sorrow, that I am not an

exception. I always believe in sheltering the members of a surveying party in houses, if possible, and this can always be done if the country is half civilized. There is nothing more conducive to getting good hard work (and lots of it) out of the men than by giving them a good, dry and comfortable shelter at night. Camping out in shelter tents in summer is very pleasant, but when fall and winter come the best thing to do is to get in a house. A good point made by Mr. Lynch is to carry the men to and from the work with teams. I am heartily in accord with him on this point, but it is very dangerous ground on which we walk. The old pioneers in railroad location never had these facilities, and those who are now chief engineers of railroads spurn such a practice, and term it weakness, laziness or dude occupations, and lament the deterioration of the profession. I have always found decided objections by the chief engineer (if he were an old pioneer) to pay for the hire of teams used for such purposes until I had shown conclusively the greater amount of work done by the men, but there are some men who prefer the old-time lumbering stage to the modern express train. My first experience in railroad location was west of the Mississippi River. We camped out in shelter tents from September, 1870, to May, 1871. There were 18 in the party, including two teamsters. The principal assistant engineer had charge of the party. We were routed out of our tents long before daylight, and those who did not respond to the call promptly had the roof drop down on them in quick order. By the time breakfast was disposed of and two sandwiches pocketed for lunch, the start was made for the work, and as a rule it was hardly dawn.

Frequently we had to walk eight miles to our work, then work till dark, never stopping to eat lunch, but snatching a bite, while the transit-man "moved up," until the lunch was finished. I acted as slopeman or topographer and, as the running of the survey lines on preliminary was very rapid, I had to keep up a continuous dog-trot all day, and consequently the lunch was dispatched in rapid transit style. When the work was done for the day, the men were thoroughly exhausted, and it would take hours of weary struggling through swamps, briar and brush patches before camp was reached. The condition of the men, physically and mentally, was such, that if it had not been for a good supper, roaring camp fire, accompanied by banjo and guitar music, the whole party would have mutinied and deserted the next morning. It is wonderful how music, in front of a roaring camp fire, enlivens the men and keeps up the organization.

The beds consisted of straw or hay, obtained from widely scattered farms, laid loosely on the ground, on which were placed two gum blankets followed by as many army blankets as the occupants felt like buying. Two men, occupying one bed, were compelled to buy their own blankets. Frequently when the party got into the new camping

ground, several hours after dark, the teams had just arrived, but no tents were set up. Every man had then to turn in and shovel away the snow, clear the brush and put up the tents; and as no straw or hay was in sight, brush of any kind, or small limbs cut off from trees, were used for the foundation of the blankets. Many a night I climbed up trees with a hatchet to cut down a feather bed. During severe storms, winter or summer, "Tenting on the Old Camp Ground" is much finer when rendered in a cosy drawing-room than out on damp Mother Earth.

Hence, I say, while it may be a "one-horse arrangement," as Mr. Lynch says, to stop at farmhouses, I would prefer to be classed as an advocate of such an arrangement and be comfortable, than to camp out and undergo all kinds of hardships.

Preliminary Survey.—In running the preliminary lines Mr. Lynch says that he uses the vernier exclusively, for ascertaining the direction of his lines.

I consider this exceedingly bad practice, and under no circumstances would I, as chief engineer of a railroad, permit any assistant to run such a line. I know of no reason why the transit-man should not make an error occasionally in reading the amount or direction of angle turned and of recording the same wrong in his note-book. Severe and long experience has taught me that the only practically safe rule or method is to use both vernier and needle, keeping calculated and magnetic courses in the transit-man's note-book, and checking the direction of the angle turned, by compelling the following level and topographical parties to enter in their note-books the turning points, noting whether the line diverges to the right or left. Any one who has tried to develop a large section of country, by means of traverse lines in all directions, connecting them again with the base line, has, no doubt, discovered that there cannot be too many ways of checking the courses and distances.

Taking Topography.—My first experience on surveys was on topography; I was called a slopeman, and comprised the whole of the topographical party. The country was rolling prairie, when away from the river, and varying from flat to steep hill sides along the river. I carried a 50-ft. tape and Locke or hand level. My eyes were practically 5 ft. from the ground, so I took levels to the right and left of the line in heights of 5 ft. I could tell, by keeping my left eye open, whereabout on the ground the cross hairs in the level, as seen by my right eye, apparently pierced the ground. Then, by stepping if the ground was gently sloping, or measuring with tape if steep, I would get the distance in which the ground rose or fell 5 ft. My notes would then be entered thus—

CENTER.

$$\text{etc., etc., } -\frac{5}{60}-\frac{5}{40}\ \Big|\ +\frac{5}{50}+\frac{5}{60}\ \text{etc., etc.}$$

The right-hand side showing plus or rise 5 ft. in 50, etc., the numerator indicating the rise or fall as shown on the left by minus sign, and the denominator the distance. In using the tape line, I always put the ring of the tape in a peg or twig stuck in the ground, then measured back to the point where I had taken observations; and in going down hill, I put the ring in a peg and walked down hill to a point where my level showed I was 5 ft. lower; read the horizontal distance on the tape, then jerked the tape off the peg, and repeated this process till through. This is without question a very crude way of taking slopes, but it is wonderful how accurately a careful and painstaking slopeman can take the topography in this way. My practice is to have the topography party consist of three men. The man in charge keeping notes, sketching streams, etc., and directing the work; while the leveler in the party carries a 5-ft. staff with a V notch cut in the top, on which he rests his Locke level when sighting and holding the box end of a 50-ft. tape, and reads the horizontal distance to the third man, who has a 15-ft. pole divided into feet and halves and the ring end of the 50-ft. tape. I also have the rodman of the level party copy the elevations of each station on a slip of paper, and every 1 000 ft. or so put this slip in a slot in a stake, made with his hatchet, for the use of the topography party, who then, knowing the elevation of each station, can pick out contours exclusively, which simplifies the draughtsman's work very much and renders accuracy more certain. The slope or topography notes must be transferred to a duplicate book every evening. This book always remains in the office or headquarters, and is for the use of the draughtsman to make up his maps the next day; in fact, it is a rule I invariably follow, that all survey notes, transit, level and topography, must be transferred every evening to duplicate books, which are called office books, while the others are termed field books. This is essential to ensure success in any operation on a large scale, for I have known on a number of occasions where field books were lost or completely ruined by falling in streams while crossing. In the coal regions when a book falls in a stream it comes out as black as the ace of spades.

Maps.—Having now run the preliminary line for one day and recorded all the notes in duplicate in the office, the draughtsman takes up the thread and prepares the maps for use in location. I always use for this purpose smooth white paper, termed "Royal," size 19 x 24 ins., and in a comparatively level country use a scale of 400 ft. to 1 in., and in a rough country 200 ft. to 1 in., and always plot the contours in 10-ft. variations. Mr. Lynch recommends a scale of 800 ft. to 1 in.; for my part, I do not see how he can use such a scale and do accurate work unless the country is very flat.

Locating.—When a considerable distance is run by the preliminary party and the maps made up so that I can glean sufficient informa-

tion as to grades and curvature, I block out a paper location in pencil on these sheets or maps and make a profile, and make changes on my location in pencil where my profile requires it. Then, if the railroad is such as Mr. Lynch writes about, 250 miles long, I organize a separate party and call it a "locating party." I provide the man in charge with a wooden box covered with leather or oilcloth, which is carried by the rear flagman, with a strap, while in the field. In this box I have a sufficient number of the aforesaid sheets or papers on which the paper location is projected, a parallel ruler, paper, wood or celluloid curves, two triangles, a straight-edge and thumb tacks. The man in charge then locates on the ground the line blocked out in pencil on the sheets, keeping the line correct by checking on the ground at intersections with (or measuring distance to) the preliminary line shown by the stakes in the ground. If the ground (at any point) is found different from what the contours of the map show, his duty then is to fit the line to the ground at that point, but always with a view to "getting on" to his line again ahead as projected on his sheets; for this purpose the different articles in his box are brought into use. Mr. Lynch says he only projects tangents on the maps of the preliminary surveys, and any good engineer can fit the curve on the ground, and condemns what he calls "a cast-iron location projected on the preliminary maps in the office." I claim that there must be a head to any successful enterprise, whose duty is to block out carefully the work of each department, which must be faithfully carried out by each man as laid out, varied occasionally as the circumstances demand; but if each engineer runs the line to suit himself, there will be a location resulting in nothing but "crazy-quilt" work. There are a number of engineers who spurn to run a location from a projected paper location, claiming they can do better by "the eye." My experience has been that such men, as a rule, are very expensive to the company employing them, both for location and in the subsequent operation of the road. There are, of course, exceptions to this rule as to every other.

I have frequently blocked out work, as mentioned above, for four and six parties at a time, and thus knew just what was being done all the time, and there was a uniformity of action throughout the entire service which could not be obtained in any other way.

The notes of the located line are also transferred to duplicate or office books every evening, and from these notes maps of the same size sheets as previously mentioned, on a scale of 100 ft. to 1 in., are made, with 5-ft. contours.

These maps can be used for blocking out a close revision of the located line, and also for the purpose of laying out the right of way or property to be bought. I always block out the right of way in pencil on these sheets, then take a roll of tracing cloth and split it in two lengthwise. I then have a roll about 18 ins. in width. This I place

on a series of sheets tacked together, trace the located center line and the property or right-of-way lines.

This roll then constitutes my right-of-way map and is my record. Whenever I wish to use a map to attach to a deed, etc., I can always make a separate tracing from the 100-ft. scale map or make a blue print from the 18-in. roll.

Broken-Back Curves.—I agree with Mr. Lynch on broken-back curves. They are of ancient history, and there can hardly be a case where an intermediate curve cannot be placed so as to avoid, not only the unsightly appearance of the line, but the rough riding on trains.

Compensation for Curvature.—Mr. Lynch recommends 0.04 ft. per degree of curve for compensation of grades. I think this a little too high, and would recommend 0.03.

I constructed the Pottsville and Mahanoy Railroad in the Schuylkill coal regions in 1886, and on this line there is a continuous grade of 165 ft. to a mile for nearly four miles. There are numerous sharp and long curves, the maximum being 12°. The compensation for curvature was 0.03 ft. per degree. After the road was completed and in operation, I frequently rode on the locomotives pushing empty coal trains up over this grade. From my own observation, and in conversation with the engineers, I ascertained that the curves required no more work from the engine than the tangents; there was a uniform push all the way up. From this I judge I had about arrived at the proper compensation.

Vertical Curves.—Mr. Lynch seems to have the same distance established from the intersection of two grades for the commencement of the vertical grade, regardless of what the grades may be. I do not think this rule a very good one. I never plot the vertical curves on my regular profile, but take a sheet of cross-section paper—scale 10 ft. to 1 in.—on which I plot the two opposing grades; then I plot an easy curve with a paper curve, and ascertain the elevations at each station intersection. This is a graphical method, and is accompanied by no error in calculation ; is simple and can be varied as much as desired.

Slides.—There is one point in railroad location not covered by Mr. Lynch. That is the knowledge the locating engineer should acquire in going through a country as to the stability of the material along hillsides. If the material overlying rocks at an angle is saturated with water and the foot of the slope is cut into, it is more than likely there will be more material come down from the hillside than was originally in the cut. The only safe way is to have a small embankment, if possible, rather than to cut the slope. My first experience of this kind was in the Monongahela Valley. There was a bank of 60 000 cu. yds. and a cut of 40 000 cu. yds. in close proximity. This necessitated a borrow of 20 000 cu. yds. I made a borrow at the same time I took out the cut. The result was, before the cut was done,

I had to waste about 20 000 cu. yds., as the hill above came sliding down into the cut.

Michael L. Lynch, M. Am. Soc. C. E. (by letter).—The writer is pleased to learn that the paper under consideration has evoked such general discussion from the many engineers competent by training and experience to throw light on this very interesting and important subject, as also from the scientists who discuss certain laws governing the formation of the earth's surface, which necessarily have a bearing on the general subject. As there seems to be some doubt or misapprehension of the writer's meaning on certain points, possibly through lack of clearness in the original paper, or difficulty in extracting its full meaning from an abstract, he will endeavor to answer the questions developed during the discussion and explain more fully.

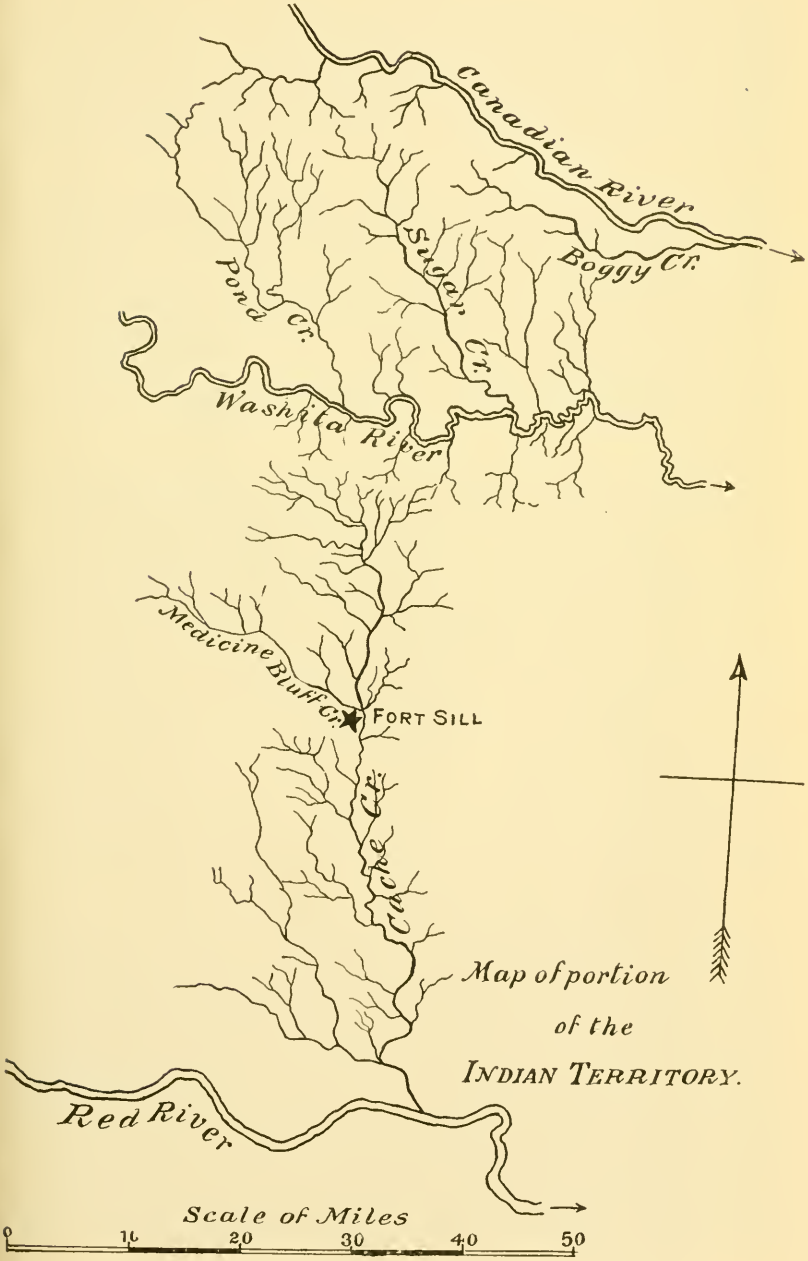
Where Mr. Moore dissents from the writer it probably occurs through some of the above causes, as their ideas are practically the same in all important particulars. The writer's meaning in regard to re-projecting and re-running lines can best be arrived at by considering what precedes it:

"The tangents and curves of the proposed location being first projected on the carefully prepared topographical map of the preliminary lines, before laying off in the field; it often happens, however, that the location so projected can be improved, and the best practice, in the writer's opinion, is to recognize the possible necessity for such a change and be prepared to make it in the field when necessary."

The projection of the location here referred to would, from the nature of the case, necessarily be a calculated one based on the topography as developed by the contours of the preliminary map and the elevations shown on the profile of the preliminary line. By the aid of this projection the location is spotted on the map, and the location of both curves and tangents determined, including intersection angles and the beginning and ends of curves, from which a very close approximate profile of the "paper location" thus projected can be made. This assumes that all the work has been accurately executed, and is the scientific location which it is intended to run in on the ground, and which should be so run in, unless through some inaccuracy of chaining, instrumental work, topography or drafting, some modification that would improve the line became apparent. In this event the paper location should be departed from and the line laid where it would actually fit the ground best, and the engineer should be prepared to re project and change that part of the line in the field. All engineers of experience on this class of work can no doubt recall many instances where such modification had become necessary, even on paper locations projected by themselves and where it would have been folly to adhere rigidly to any calculated cast-iron location project where it was found in error.

In regard to the methods outlined by Mr. Storey, the writer agrees with him as to the necessity for accurate work in every department, and believes that the main inaccuracies in such work occur in the chaining and topography; this is evidently implied in his description of the method of chaining and of the engineers noting the topography in person on a skeleton plat without contours. The writer is of the opinion, however, that the use of a skilled topographer developing the contours in the usual manner, which are afterwards platted on the preliminary map, is the better plan, as the locating engineer is supposed to have a detailed knowledge of the ground which he utilizes in connection with the contour map in projecting the paper location. Requiring the locating engineer to perform the work of the topographer would necessarily have a tendency to delay the party, by taking his attention from the general direction of the work while performing the duties of a subordinate. The fact that "every half mile or so a check is given, to be taken on the preliminary" is a frank and proper admission of the possibility of error, even in such a carefully conducted survey as Mr. Storey describes, and it seems to the writer that the "machine" he mentions would be liable to "slip an eccentric" in trying to remedy the error otherwise than by intelligent direction on the ground. The writer is not prepared to admit that the methods described by him are expensive. An expensive location is one that is blunderingly executed where a poorly equipped party is put in the field without allowing the locating engineer sufficient time to properly develop the country by a thorough reconnaissance, when it becomes a case of go ahead and back up, until after a certain lapse of time some kind of a line is adopted without any definite knowledge of how it would compare with any other lines in the country. The use of an odometer would certainly give satisfactory results where the country was favorable, and would no doubt be quickly adopted by the engineer. In matters of this kind, transportation of supplies, etc., the engineer is governed by circumstances in general and the character of the country in particular. Often it may become necessary for the greater portion of a reconnaissance to be made on foot.

There seems to be considerable difference of opinion regarding the existence of a general law in connection with the smoother or rougher slopes of a valley. The writer simply noted what came under his own observation, and is fully convinced that such a law exists. By the term valley, he desires to be understood as meaning the slopes on either side approaching a stream, and not alone the contracted space between the foot of slopes. Mr. Kelley, while not fully agreeing with him, cites several instances in corroboration, and in those instances where the conformation is obscure, the streams flow in a southeasterly direction; this latter characteristic was noted in the original paper, its exitsence being more noticeable where the streams lay in the general direction of the meridian or at right angles to it.



Map of portion of the INDIAN TERRITORY.

Mr. Barlow agrees with the writer in regard to the north slope of an east and west valley being generally smoother than the one on the south, and Mr. Maxwell's experience in Washington, Nevada and California leads him to agree with the writer, and while Mr. Brendlinger never had the subject brought to his attention before, he cites several important roads as having their "maximum grades on the south and east."

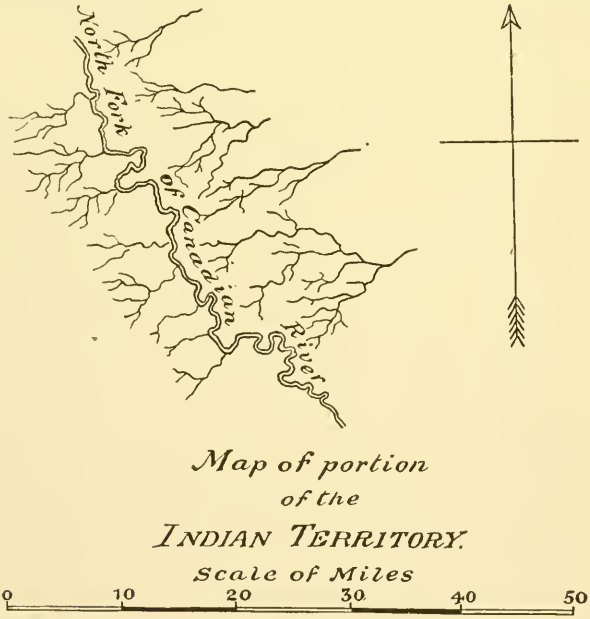
The writer is confident that careful observation will develop the existence of this general law more marked in some cases than in others, always excepting the traditional exception which goes to prove the rule. It is rather a coincidence that all of the three gentlemen who discuss the subject from a scientific standpoint, Messrs. Tarr, Davis and Willis, agree as to the non-existence of such a law, and Mr. Crowell is not quite clear as to the writer's meaning. Probably the sketch (page 127), taken from a sectionized Government map of the Indian Territory, will help to convey the author's meaning more clearly.

Referring to the sketch, the distance between Red River and the Washita is, in round numbers, 65 miles, and of this distance, to the head of Cache Creek, about 60 miles is in the drainage of the Red River, and 5 miles in the drainage of the Washita, resulting in a very steep descent to the latter stream. Continuing north across the Washita along Sugar Creek for about 30 miles to its head we find ourselves within 4 or 5 miles of the Canadian River, to which the descent is consequently very steep. This is a good example of the relative declivity of slopes between streams, and illustrates the writer's meaning where he says that "the engineer will find it more difficult to get a line down into a valley from the south than in leaving the valley going up the north slope." Again, the portion of the north fork of the Canadian River flowing in a southerly direction (see sketch, page 129), taken from the same map, illustrates why it would be "more difficult in getting down the east slope of a valley than in getting up the west slope."

The writer's attention was first drawn to this peculiarity several years since while engaged on a reconnoissance for the Gulf, Colorado and Santa Fé Railway, extending from Northern Texas up through the Indian Territory to the Kansas line, the general direction being across some of the largest streams in the Southwest, where he found this law fully exemplified, with one noticeable exception. He may also mention another very noticeable case of this law relating to streams flowing in the general direction of the meridian, which came under his personal observation in locating the Shreveport extension of the St. Louis Southwestern Railway, between Louisville, Ark., and Shreveport, La.

In this case, the line occupied the country between two streams, both flowing about due south, the Red River on the west, and the

Bayou Bodceau on the east. Upon reconnoissance, it developed that the divide lay close up to the Red River, along its left bank, with such steep, precipitous and broken slopes towards the river that a line was utterly out of the question ; whereas, by swinging the line further east towards the drainage of the Bodceau, a tangent 22½ miles in length was obtained with light work. No doubt, certain cases may exist where these conditions do not obtain, or may be reversed; possibly the drainage of the Yellowstone River, in Montana, would be a case in point, but the writer believes them to be rare, and in very broken mountainous country, where serious geological disturbance has taken place, drainages, divides and slopes become mixed up in such inextricable confusion that no law seems to govern, so that, upon whichever side of a stream the engineer may find himself, he is sorry that he is not on the other.



The valleys proper of streams partake, to a certain extent, of the character of the slopes extending back to the divide, it being self-evident that steep bluffs are liable to be found on either side of a stream, depending on how the stream caroms from one side of the valley to the other. Again, some streams have very little valley, partaking more of the nature of a gorge or cañon, in which case the law would not apply as to their valleys. This is a peculiar phase of the subject, and properly belongs to the domain of abstract science rather

than to practical engineering, and is worthy the attention of scientists. The engineer simply notes the fact and takes advantage of it.

The main advantage to an engineer of the knowledge of this law is, that it would direct him on reconnaissance to where he would be liable to find the smoothest ground, and enable him, beforehand, to form an intelligent opinion of a project in general.

A glance at any good map will show many examples of where this condition has been taken advantage of. The Union Pacific keeps along the north bank of the Platte, and where it diverges at Julesburg, one branch keeps up the north bank of Lodge Pole Creek. The Burlington and Missouri Valley road keeps on the north bank of the Republican River, and where it diverges at Culbertson, one branch keeps up the north bank of Whiteman's Creek. The Atchison, Topeka and Santa Fé road keeps on the north bank of the Arkansas River for the greater portion of the distance along that stream. Many other notable examples could be cited, but, of course, when the question of competition and the building of branch lines comes in, a road is liable to occupy either side of a stream.

The discussion by Mr. Ainsworth touches certain points which the writer did not expect to see come up, but which may, nevertheless, be considered pertinent to the subject. It seems, to the writer, that a broad general principle is here involved, and that the way of the engineer is plain. A syndicate or company desires to build a road and employs an engineer; he is engaged under a salary agreed upon, and they are entitled to his best service. They naturally consult him as to the character of line available over a certain country, upon which he advises them; it is not necessary for him to know all the details of their plans, financial or otherwise. They instruct him as to the amount of funds available, which necessarily governs the character of the line, both as to location and construction; he goes ahead and gets the best line available under the conditions, and his work is done. Of course, it may devolve on him at some stage of the proceedings to try to save them from themselves by endeavoring to head them off from some piece of folly and adopt such a line as his better judgment would dictate, but if his suggestions are not adopted, and he receives definite instructions as to their requirements, then he is bound to follow them; provided, always, that no point of honor or ethics is involved, or that the professional work required of him would be of such an inferior character that he would not wish to have his name identified with it.

As to secrecy, of course, the locating engineer should consider that he occupies a position of trust, and has to guard against the professional town boomer and land shark as well as against the avaricious landowner. Right here is where one of the valuable features of a horseback reconnaissance comes in, as it can be performed without disclosing one's intentions to land speculators or competing railroads; as in

ordinary country the general character of a line, including maximum gradient and curvature and approximate estimate of cost, can be determined and an intelligent report furnished, thus enabling one road to take all necessary preliminary steps towards occupying a country without unnecessary warning to a competitor.

Mr. Purdon's discussion contains a series of aphorisms which are sound location doctrine and very much to the point. There is no doubt but that serious mistakes have been made by skirting the foothills of a slope too closely, resulting in a series of alternate light cuts and fills; the resulting light work being liable to washouts from the rapidly descending waters of the slope in heavy rainstorms. Better, as he says, to lay the line on lower ground on moderate embankment and with ample borrow pit or ditch on the up-hill side, to catch the drainage of the slope and convey it safely to the nearest opening. The method of taking topography described by him as used by Mr. Van Sant is new to the writer, and seems to be an ingenious and effective method.

The writer did not mean to convey the idea that general principles should not govern in railroad location as Mr. Beckler and Mr. Brendlinger seem to have inferred. The original paper states that, "in regard to curvature and grades, it is assumed that a certain maximum has been agreed upon which cannot be exceeded," it being understood without saying that no intelligent comparison of two or more lines through a country could be made without considering all the elements that would enter into the completed road as turned over for purposes of operation, including the character of curvature and its amount, character and length of gradients, rise and fall, percentage of curvature and tangent, character and amount of grading, bridging, etc., together with an estimate of cost; in fact, he considers no location profile complete without an estimate on the face of it for each section. He meant to convey the idea, that, as countries and conditions differ, the engineer should be prepared to meet different conditions as they exist, without being governed by inflexible rules. Of course, it should go without saying that no comprehensive system of location could be carried on without intelligent direction and supervision by the chief engineer. In regard to the matter of vertical curves on summits, the writer believes that the method suggested will give curves of sufficient length, even upon steep gradients. Take, for example, two 2% gradients meeting at a summit; under the rule, the ordinate for correction at the apex would be 2 ft., and for the 1st, 2d and 3d stations on either side of the apex the ordinates would be 0.50, 0.13 and 0.03, respectively, the 4th station on either side being the beginning and end of curve, which would result in a curve 800 ft. long, very flat at the ends, it is true, but which, if considered too short, could be modified by the substitution of the method illustrated for two descending

gradients, with sketch of level between. Platting the above to a large scale on cross-section paper will give a good idea of the resulting curve.

The use of the stadia and plane table has been advocated by several gentlemen taking part in the discussion. The writer has a high opinion of the stadia instrument, having used it on United States Government river surveys, and believes it could be profitably used on preliminary railroad surveys, under certain conditions, as distance, azimuth and elevation can be determined by its use. The plane table, of course, is valuable in filling in the details of topography, but, in the writer's opinion, is not needed on railroad surveys. In this connection it should be kept in mind that a detailed accurate topographical map of the entire country is not the object sought, and railroads should not be required to spend money for such a purpose, the main object in view being to get a line on the most suitable ground for the building of a railroad. In accomplishing this result, a great deal must depend upon the individuality of the engineer and his skill in judging country, his mental capacity for absorbing and assimilating a knowledge of what he sees, and of his memory in keeping connected track of the topographic features of a country and utilizing them to the best advantage for his purpose.

In the matter of railroad location, there may be said to exist two schools of extremists, one of which is led by the man with cross hairs in his eye, who would scorn to use a hand level or hand compass, and who pays but the slightest attention to establishing contours on his preliminary map, depending mainly on his eye for results; the other is led by the theoretical crank, who wants to cross-section the whole country with all sorts of instrumental observations, insisting on having everything done according to pet theories regardless of actual consequences. The latter practice has a tendency to narrow and dwarf a man's perceptive faculties, rendering him a species of machine, very slow in operation and dull of perception in grasping the salient features of a country and taking advantage of them, and comparatively helpless when unusual conditions present themselves; while the former will never achieve the best results until he frankly acknowledges to himself that he doesn't know it all, and is willing to call to the aid of his natural judgment the portable instruments and appliances designed for the purpose. A word about treatment of parties in the field, as referred to by Mr. Brendlinger. The writer can sympathize with him, having once been a member of a party which was usually routed out in the latter part of the night, had breakfast eaten and tents down, ready to strike out on foot through the gray dawn for the line, often 4 or 5 miles distant, to be regaled at noon with a lunch consisting of a few hard baked biscuits and some meat carried in a little bundle attached to the buckle strap of each man's pantaloons, the weather being so

extremely hot that no pockets were available owing to lack of wardrobe; then back to camp after the day's work, another long tramp, reaching it after nightfall to a late supper, or, as one of the men wittily put it, "We had two suppers the same night." Such treatment of men is unjustifiable and unworthy of an engineer, and anything but economical or subservient of good work or discipline. The writer is, however, in favor of a party having its own tents and camp outfit. It is much more independent and can do more work, especially in sparsely settled regions, where the accommodations are usually limited and the houses generally small. The writer has more than once been entertained while on reconnaissance where accommodations were so limited that the host, his wife and children and the guest had all to occupy the same sleeping apartment, and he is confident that Mr. Brendlinger and other engineers who may have been similarly entertained will join with him in extolling such as the highest type of hospitality.

Mr. Brendlinger misunderstands the writer in regard to using the vernier alone on preliminary work; the needle should be used as a check by all means, both on preliminary and location.

In regard to taking topography, the various methods described are good, but a good topographer is usually the most difficult man to procure for a party. It seems strange that the engineering schools do not pay more attention to practical field training in this branch through a summer school for field practice during vacation, or at some other suitable time. It is an interesting study, and one to which young men usually take with pleasure.

The question has been asked as to the writer's meaning where he uses the term settled weather in connection with barometric leveling. He means settled weather meteorologically, that is to say, when no atmospheric disturbance is taking place that would interfere with the reading.

The question of bridge location has been fully discussed by Mr. Greiner. Of course, the more the locating engineer knows about bridge construction, and general construction for that matter, the better, but it does not necessarily follow that he should be a bridge expert in order to properly locate the best crossings on a line. The original paper recommends the advisability of locating the best crossings on the important streams, and making the intermediate sections of the line subordinate thereto. The engineer will naturally endeavor to get as square a crossing as he can for every stream, large and small, and for the more important streams take duly into account the nature of approaches, character and stability of banks and bed of stream, and general course of the stream above and below the crossing, and will be careful to show the various high-water marks of previous years on his profile, but, as is well known, the estimate of bridging, like every-

thing else shown for each section on a location profile, is only approximate. The determination of the class and character of structure, general details, etc., is the province of the chief engineer and bridge expert.

The subject of railroad location is a very broad one, and volumes could be written upon it. The discussion and description of methods elicited, while many of the latter are necessarily of old application, will, it is believed, add somewhat of interest to the literature of this very important subject.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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691.

(Vol. XXXI.—February, 1894.)

THE WATER WORKS OF DENVER, COLORADO.

By James D. Schuyler, M. Am. Soc. C. E.

Read September 20th, 1893.

WITH DISCUSSION.

In order to present intelligently the interesting features of recent construction in the new water works of the city of Denver, with which the writer has for three years been connected in the capacity of consulting engineer, it may be of interest to give first a brief historical account of the development of the old works and thus explain the conditions which led to the construction of a new system.

The first works, built in 1871, consisted of a pair of Holly pumps, with a capacity of 2 500 000 galls. daily, drawing water from a large well sunk in the gravel beds of the Platte River, at the foot of Fifteenth Street, then the main thoroughfare of the city. The town had then but 6 000 inhabitants, but was growing rapidly, and two years later an additional pump with 2 000 000 galls. capacity was added, which sufficed for public needs till 1879, when a higher source of supply was sought above the sewage contamination of the spreading city. The new works were located in West Denver, two miles above the original location, where it was possible to utilize the water-power of

the river for pumping purposes. A canal, some two miles long, so wide and capacious as to be dignified by the name of Lake Archer, was constructed from the river to the new works, where a vertical fall of 30 ft. was available. A battery of turbine wheels and a set of four Holly horizontal pumps, having a combined capacity of 5 000 000 galls. daily, constituted the new plant, to which were added two more Holly pumps with 5 000 000 galls. additional daily capacity, in 1881. The water-power being insufficient at times for the increased duty, a 350-H. P. compound engine, with corresponding boilers, was also added at the same time. Both sets of pumps derived their supply directly from Lake Archer, and while the volume was adequate, the quality was not altogether satisfactory. The lake, in time, accumulated a great mass of silt and decaying vegetable matter, which began to contaminate the water, and in 1884 the water company decided to seek a purer supply still higher up the stream, by developing the underflow in the gravel beds adjacent to the river, on the opposite side, and 3¼ miles above the works. There were then no factories or settlements of material size above the intake, the military post had not then been located, and the city had not grown to within threatening distance of the source of supply; hence, the site selected was a good one for the time being. The development of the new supply was planned and executed by Mr. C. P. Allen, chief engineer of the company, and was the originating motive for the evolution of the wooden stave pipe patented by him, which has since come into extensive use. The grade from the gravel beds to the works was so slight that a very large pipe was needed to carry the volume of water required, and the maximum pressure upon it was so low that there seemed to be an especially good field for the employment of some material very much cheaper than iron. It was finally decided, after considerable experimentation, to lay a wooden stave pipe, 48 ins. in diameter, banded with round rods, the top half to be of California redwood and the lower half of Colorado pine. The cribs or galleries which gather the water and feed the pipe were made of rough lumber, 4 ft. in width, and 1 to 4 ft. high, with open joints, the tops of which are some 6 ft. below the level of the low-water surface of the river. Some 1 400 ft. of these cribs were placed, and resulted in a minimum yield of 12 000 000 galls. daily, which increased to 18 000 000 galls. during flood stage. The water, filtering through gravel, was clear and pure, and when the works were

completed there was a vast improvement over pre-existing conditions. The pipe terminated in a 12 000 000-gall. reservoir at the works, whence it is pumped into the mains. This still constitutes the principal source of supply of the Denver Water Company, although the yield of the cribs has been very much reduced by the waste from a large paper mill that pours its sewage into the river a few hundred feet above them, forming a film of paper pulp over the bed of the stream that interferes with the free percolation of water through the gravel. To remedy this defect, in a measure, deeper cribs were sunk next to the channel of the stream in the beginning of 1892, and a centrifugal pumping plant of large capacity was erected to lift the supply some 10 to 15 ft. into the wooden conduit. This arrangement necessitates a double pumping of the supply; and it is only a temporary remedy, as the strong draft from the new cribs has been gradually choking the gravel to a greater degree; and whereas, at the outset, the centrifugal was only needed during low-water stage in the river, it has of late been run without cessation.

The success attained in developing the underflow of the Platte encouraged the company to attempt a similar development on Cherry Creek, a tributary stream that joins the Platte River in the heart of the city. This stream heads in the Great Divide, south of the city, and has a minimum slope of 30 ft. per mile. It has a large water-shed area (380 sq. miles) of plains or prairie lands, and a wide, deep bed of sand, and is subject to torrential freshets, but has a very small normal flow, as it has no mountain drainage and is not fed by perennial snows. Its slope rendered it practicable to bring out its underflowing waters in a comparatively short conduit and deliver them to the city mains at a sufficient elevation to avoid the necessity of pumping. In the winter of 1886-87 a wooden stave pipe, 37 ins. in diameter, five miles long, was laid to tap a system of semi-circular cribs sunk in the bed of the creek at a maximum depth of 25 ft. below the surface, or about 22 ft. below the normal water surface. These cribs were tight on top and sides, but open at bottom, this construction being preferable in quicksand. They cross the channel from bed-rock on one side, within 700 ft. of the outcrop, to bed-rock on the other, and hence drain all the underflow to the depth to which they are laid. Borings showed the extreme width of this subsurface channel to be about 2 000 ft.; maximum depth, 41 ft.; area of sand cross-section about 62 000 sq. ft., of which about 40%

was above the level of the cribs. The lower end of the pipe terminated at a covered reservoir, excavated in earth, with a capacity of 7 000 000 galls., located on Cemetery Hill, in the southeastern suburbs of the city, the level of high-water surface being 5 417 ft. above sea level. At the time of its construction this was the limiting height of city pressure, but in 1891 the pressure was increased throughout the city some 20 lbs. (giving 100 to 110 lbs. pressure throughout the business quarter), and since then the Cherry Creek supply has also to be pumped. When the 37-in. pipe was completed, the flow reached a maximum of 9 000 000 galls. daily. After four months this began to decrease, and in one year had diminished to 2 000 000 galls. daily, which is about the normal yield since that time. This yield is at the rate of about 80 galls. per day per square foot of cross-sectional area drained by the cribs, or about 2 500 galls. per square foot of bottom area of the galleries or cribs.

The quality of this water has always been better than that of the Platte, but both have been affected by the largely increased area of irrigated land, the seepage from which, returning to the stream, carries with it a strong solution of alkaline salts that render the water hard and objectionable to the taste. These streams of seepage water are so considerable in volume that, according to recent gaugings by the State Engineer of Colorado, the water returning to the Platte in this way above Denver amounted to 96.38 cu. ft. per second in November, 1891, at the close of one irrigating season, and in March, 1892, at the beginning of the next, it was 84.57 cu. ft. per second, ranging from 47 to 56% of the total flow of the river at the time of the gaugings. The water is frequently darkly stained, a brownish color, especially during the irrigating season, and in that state is brackish and unpleasant to the taste; in fact, quite unfit to drink until it mingles with the larger volume of the river. Irrigation is indispensable to the welfare of the country, but it is frequently accompanied by this disadvantage, that it does not improve the quality of potable water where it is practiced extensively.

In 1890, The American Water Works Company, successors to The Denver Water Company, began active operations with the object of meeting the competition of The Citizens' Water Company, organized the preceding year, by a large extension of its mains in the city, an increase of its pumping capacity by the erection of two high-duty Gaskill pumps of 12 000 000 galls, daily capacity, and the development

of an additional water supply. The scene of operations for the latter work was on Cherry Creek, some distance above the head of the 37-in. pipe, where, during the summer of 1891, some $900 000 was expended in preliminary preparations for drawing a vast water supply, sufficient for all present and future needs of the city, from the sand beds of the creek. Unfortunately, financial embarrassments prevented the completion of these works before any water was developed, as their operation would have been of great scientific interest. A large timber caisson, some 800 ft. long, 16 ft. wide and 40 ft. deep, was to have been sunk across a portion of the channel, which was to have served as a receiving well, from which water was to be pumped to a large reservoir constructed on a hill half a mile distant, 150 ft. above the level of the stream. This reservoir has a capacity of some 95 000 000 galls., and was built with unusual disregard to expense, the embankments being supported inside by massive double walls of masonry, standing vertically 20 ft. high, the space between them being filled with thin cement grout. Some sixteen heavy pressure 48-in. valves are inserted for various functions in the walls. Rows of handsome brick and cut-stone dwellings for the use of employees were erected. The general scale of operations was evidently based on the expectation that this would be the chief, if not the sole, source of supply for the future, and that upwards of 50 000 000 galls. was to be made immediately available, which was the amount frequently mentioned in press interviews as the quantity to be at once obtained. Much doubt has been expressed by engineers of good repute, especially by those who have had occasion to experiment on the velocity of water percolating through sand, as to the possibility of the existence of any very considerable volume of underflow in this stream, even approaching in amount the quantity above stated, and on account of the extraordinary expenditure made prior to a conclusive demonstration of the volume of probable flow, it has seemed to the writer to be of sufficient interest to call special attention to the case.

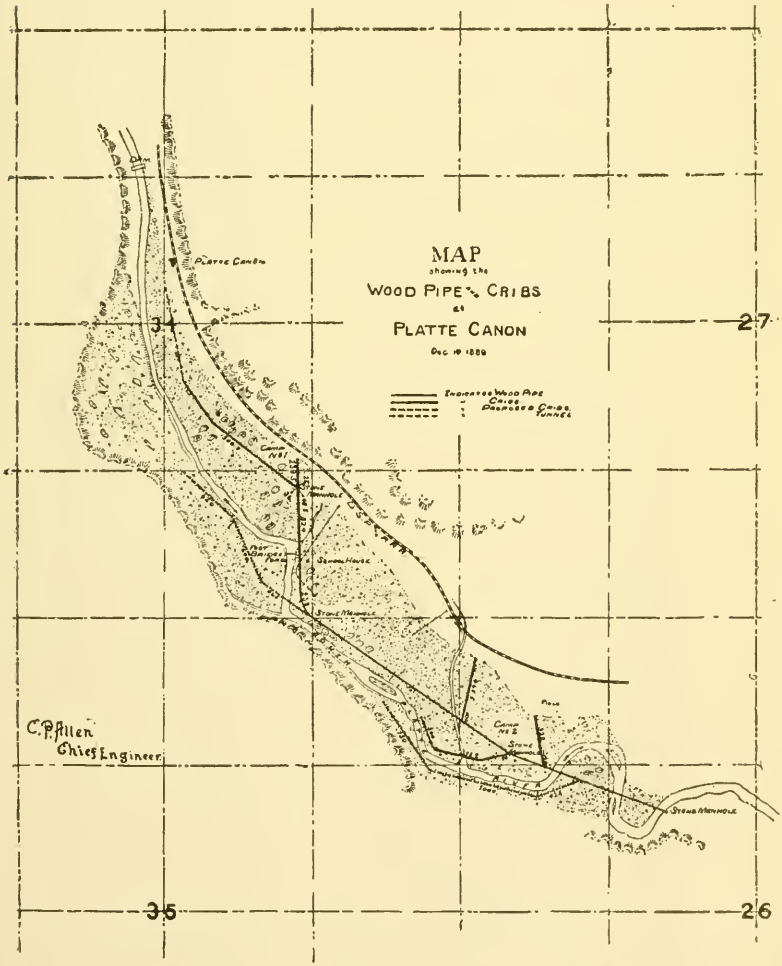
But little positive data seem to have been recorded as to the rate of underflow or percolation through sands on various slopes, but all notes on the subject that the writer has been able to collect seem to agree fairly well within limits of from $\frac{1}{2}$ mile to 2 miles per annum, the velocity being apparently influenced less by the slope than by the degree of fineness of the sand. Applying the maximum of these veloci-

ties to the case in point, the cross-sectional area of sand required to pass even 40 000 000 galls. daily, assuming the voids to be 30% of the mass, would be 610 000 sq. ft., or, say, 3 miles wide by nearly 40 ft. average depth, which is 10 times the area of cross-section determined by borings at the cribs a short distance below, and 19 times the area presented by the side of the proposed caisson, heretofore referred to. The writer has been so deeply impressed with the wide-spread fallacies entertained by many intelligent people on the subject of fabulously great and "inexhaustible" volumes of water presumed to be flowing beneath the surface of sandy beds of western streams that he wishes to provoke discussion on the subject, and elicit an expression of the experience and opinions of engineers, either in opposition to, or corroboration of, his views on this question. In his opinion, a visible, tangible water supply, stored up in a capacious reservoir, is always preferable to an invisible, immeasurable, conjectural supply lurking hidden in a bed of sand that is chary about giving it up, especially when the one is as easily and cheaply secured as the other, as was the case at Denver.

The Works of The Citizens' Water Company.

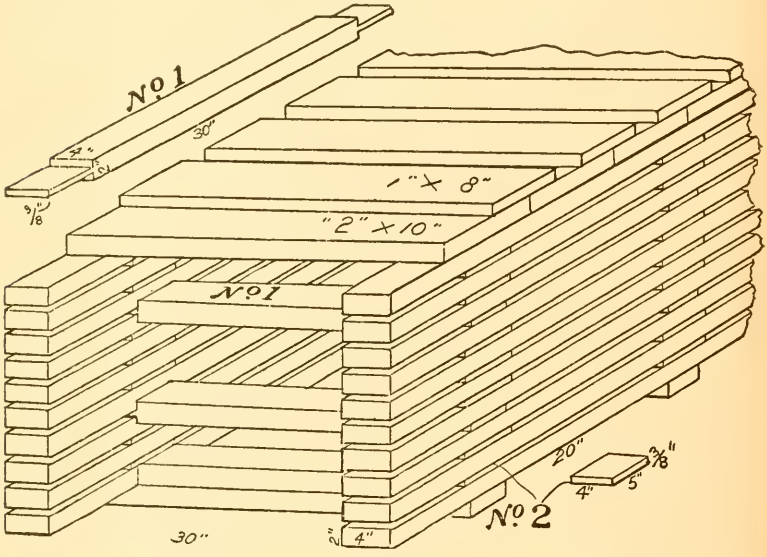
The fact that over 400 artesian wells were bored by private parties, at an aggregate cost of over $500 000, within the city limits of Denver, from the time of the discovery of this source of supply in 1883 until its limited volume was fully demonstrated some years later, is a significant commentary on the quality of the general water supply at that time, and is evidence of a desire to obtain purer water. The Citizens' Water Company was organized March 29th, 1889, by a few wealthy citizens of Denver, with the avowed purpose of meeting this public demand. Its organizers had been minor stockholders in the old water company, and had endeavored to impress the majority holders with the necessity of adopting a gravity system of supply from a pure source in the mountains. Failing in this, and being more deeply interested in the continued growth of the city than the majority seemed to be, they finally sold their stock and organized a rival company. Thoroughly conversant with the details of the business and the general problems of the situation, they entered into the enterprise with confidence in the ultimate outcome. Enlisting the services of Mr. C. P. Allen, Chief Engineer, and Richard Holme, General Manager, of the old company,

they lost no time in beginning operations, and within 15 months from the date of their organization had completed a 30-in. gravity pipe line from Platte Cañon to the city, 20¼ miles long, had laid some 70 miles of mains in the city and secured a large share of patronage, fighting their way inch by inch through injunctions, harassments and personal encounters, whose history would fill volumes.



The New Works.—The Platte River, before emerging from the Rocky Mountains and entering upon the plains, passes through many miles of rock-bound cañons, where its waters are so thoroughly aerated in

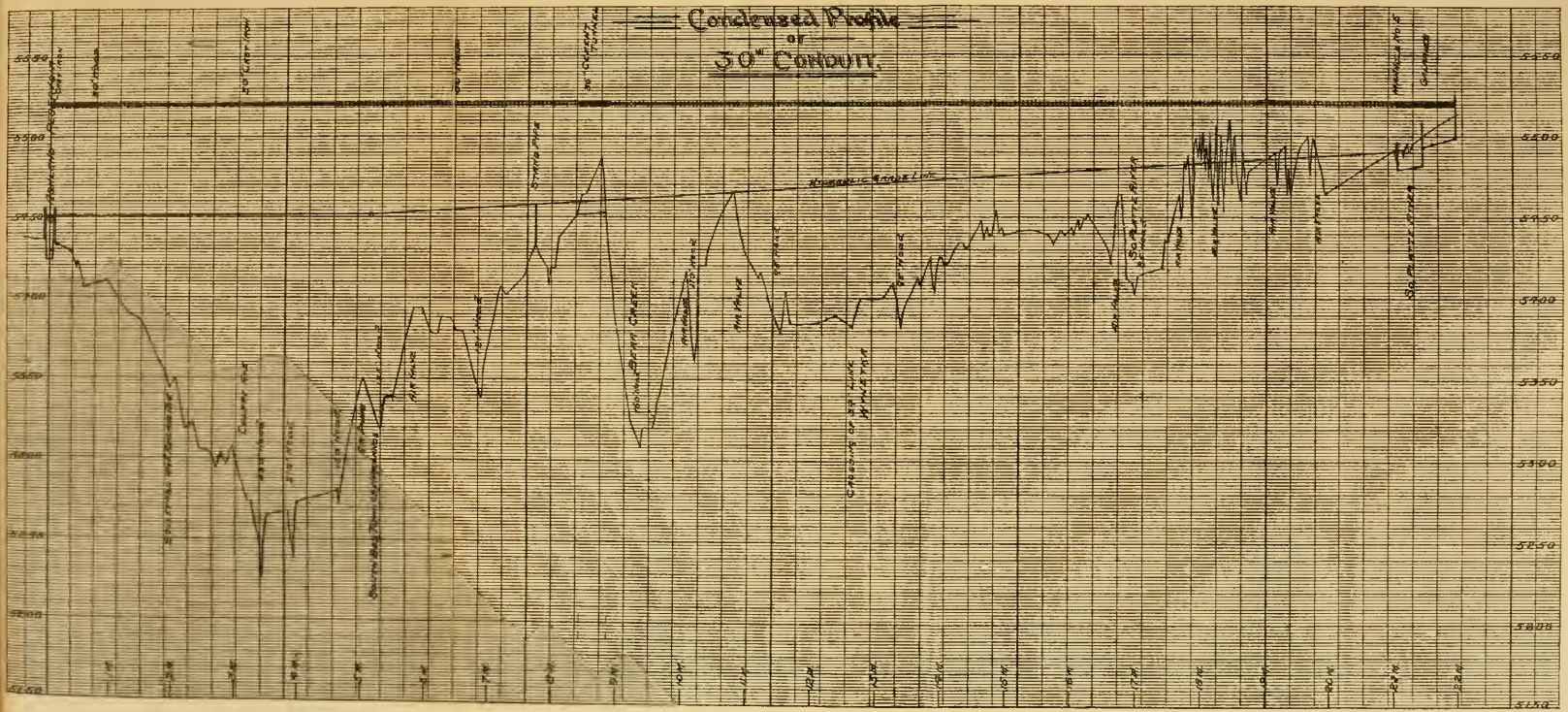
tumbling over a succession of cascades that any possible impurities that might have been imparted to them by the sparse settlements on its upper sources become neutralized and eliminated. At the point of exit from the mountains the stream has deposited great beds of gravel, covering several hundred acres, which are naturally saturated with water that sinks in the bed of the cañon above. To obtain this water for the preliminary supply, the company purchased a large area of these gravel beds and set to work to drain them. Beginning at a point whose elevation was sufficient to admit of the free flow of water therefrom to the highest points in the city limits, trenches were dug on light grades following up the general course of the valley, with several lateral branches. In these trenches were constructed rectangular wooden cribs or galleries, 30 ins. square, formed of 2 x 4-in. rough lumber on the sides, with ¼-in. spaces between them, the tops solidly planked over and the trenches refilled. The total length of main trunk cribs is 3 584



Plan of Crib-Work of Citizens' Water Co.

ft., with 328 ft. of laterals. Their depth below surface is 14 to 22 ft., and their total cost, including five masonry manholes with valves, was $47 807 90, an average of $6 89 per foot. The total excavation required was 45 212 cu. yds. The result of this work was a measured flow of

PLATE IX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 691.
SCHUYLER ON WATER WORKS OF DENVER.



3 260 000 galls. of pure, clear water, which remained constant for many months, until the galleries were flooded to increase their yield up to the capacity of the pipe—8 400 000 galls. daily. Gaugings at the different manholes indicated that the yield of the cribs at 22 ft. depth was nearly three times that at 13 ft. Four large ponds were formed over the galleries, and water turned into them from the river. They cover about 20 acres and are cleaned out twice a year, the deposited silt being scraped out and the surface of the gravel loosened by plowing. From them the water percolates freely to the drainage galleries, where it emerges perfectly filtered and clarified.

The 30-in. Conduit.—From the cribs of the Ashland Avenue distributing reservoirs, located some four miles west of the center of the city, a 30-in. conduit was constructed, the total length of which is 107 012 ft., or 20.26 miles. Of this distance, the greater portion, 87 580 ft., or 16.4 miles, is made of wooden stave pipe. Cast-iron pipe was required to be used for a distance of 17 145 ft., passing through the streets of the municipality of Highlands. A tunnel on the line, 2 288 ft. in length, 36 ins. in diameter, lined with concrete, completes the entire length. The wooden pipe is banded to withstand a maximum pressure due to 185 ft. static head below the hydraulic grade, which extreme occurs near its junction with the iron pipe. At the crossing of the valley of Bear Creek, it lies 150 ft. below the grade line for more than 1 000 ft., and at numerous points it withstands from 50 to 100 ft. head of pressure. The total number of bands used was 271 900, which, divided by the length, gives an average spacing of 3.86 ins., which is sufficient for an average head of 70 ft. throughout. This, however, was much more than was required for the grade line from the present source of supply, the extra banding having been added with a view of increased pressure to be given to the line by a proposed branch line from a large reservoir that was to be constructed at a high level near the head of the pipe.

The cost of this 16.4 miles of wood pipe was very moderate, as shown by the following detail:

1 869 000 ft., B. M., Texas pine, surfaced, at $27.50	$51 399 28
271 900 ½-in. steel bands and shoes	54 299 55
Erection of pipe, at 5.1 cents per band (contract)	13 866 03
Total	$119 564 86

This is an average of $1.36½ per foot. Trenching and backfilling cost 48.3 cents per foot, having been rendered unusually expensive by several miles of cutting from 10 to 30 ft. deep, and one river crossing, where the channel was turned and the pipe laid several feet below the bed of the stream.

The condensed profile of the entire conduit, Plate IX, will show at a glance the general topographical features of the line and the pressures borne by the wood pipe in the numerous dips below the grade line. The general course of the conduit is north, and for the greater portion of its length it is on the west side of the Platte River.

Although its actual terminus is at the reservoirs mentioned, connection is made with the city distributing system at various points from two to five miles above the reservoirs. Being thus freely attached to the city mains, with open connection with the reservoirs, only the surplus over and above the consumption at any time passes into the reservoirs, which occurs chiefly at night. Whenever consumption temporarily exceeds the volume of normal flow in the conduit, the deficit is drawn from the reservoirs, and thus for the lower two miles or more the water flows alternately into and out of the reservoirs through the conduit. This varying condition renders it impossible to fix an absolute hydraulic grade, but that which is shown upon the profile is assumed to terminate at the point where the first large lateral taps the conduit. The capacity of the conduit to the reservoirs, with city connections closed, is 8 400 000 galls. daily at the maximum, or 6 300 000 galls., with reservoirs at full stage. The measurement has been made both volumetrically, *i. e.*, by observing the depth added to the reservoirs in a given time, and by velocity measure with a current meter at a manhole in the tunnel. The results indicate that in applying Kutter's formula to wood pipe, as low a coefficient n as .0096 can be used, although we have generally assumed $n = .010$ in all calculations of capacity. The wood pipe, having a smooth interior which becomes smoother with use, has a decidedly greater carrying capacity than cast-iron pipe of equal diameter, the difference amounting to about 16% as far as deductions are warranted from observations covering a limited range of diameters.

The Second Conduit.—Within a year after the completion of the first conduit, it became evident that another of still greater capacity was immediately required to supply the growing demand for "mountain

water." The general plan proposed from the beginning of the enterprise had been to construct a conduit of 60 000 000 galls. daily capacity from the Forks of the Platte, some 15 miles above the mouth of the cañon, to a large storage and settling reservoir located within the "hog-backs" at the foot of the mountains, some two miles west and 500 ft. higher than the head of the first conduit, and thence, by a short pipe connection with the latter, increase its capacity by the addition of 100 ft. or more head, to the extent of probable requirements. It was in pursuance of this plan that the pipe was so heavily banded, as heretofore mentioned. The location of this great conduit along the cliffs of the cañon was made and work begun on the dam, but as the dam and conduit were estimated to cost about $1 000 000, and required three years or more to complete them, and no addition to the supply could be expected from any quarter until both were completed, it became apparent that the delay was too great a sacrifice of time. This led to a change of plans, and it was finally resolved to build a second conduit directly from the river, and filter the water, starting high enough up in the cañon to give a grade that would deliver 16 000 000 galls. daily through a 34-in. pipe. It was to start on the left or west bank of the stream and follow parallel with the first conduit, but on higher ground, for about half its length, then to cross the valley of the Platte through the town of Littleton and enter Denver on the highest ground to the east, on the opposite side of the city from Conduit No. 1. It was considered highly advantageous in a gravity system to place the city between two large feeders to its mains. This general plan was carried out and the conduit completed in April, 1893, although a material change was made at the head as the result of the purchase of a large natural basin, locally known as Marston Lake, to be used as a storage reservoir. This is located about nine miles in a direct line from the geographical center of Denver, and may in the near future be within the city limits. It is 1½ miles northwest of the nearest point on the line of the second conduit, where the latter turns eastward to cross the Platte. A connection was made from the lake to the second conduit by means of a 44-in. wood pipe, which was extended from the connecting point east for 2 000 ft. as a part of the main conduit, to where the latter crosses Conduit No. 1, with which it was also connected at the crossing. Thus the lake can be drawn upon to supply both lines through the one 44-in. pipe, giving to them a

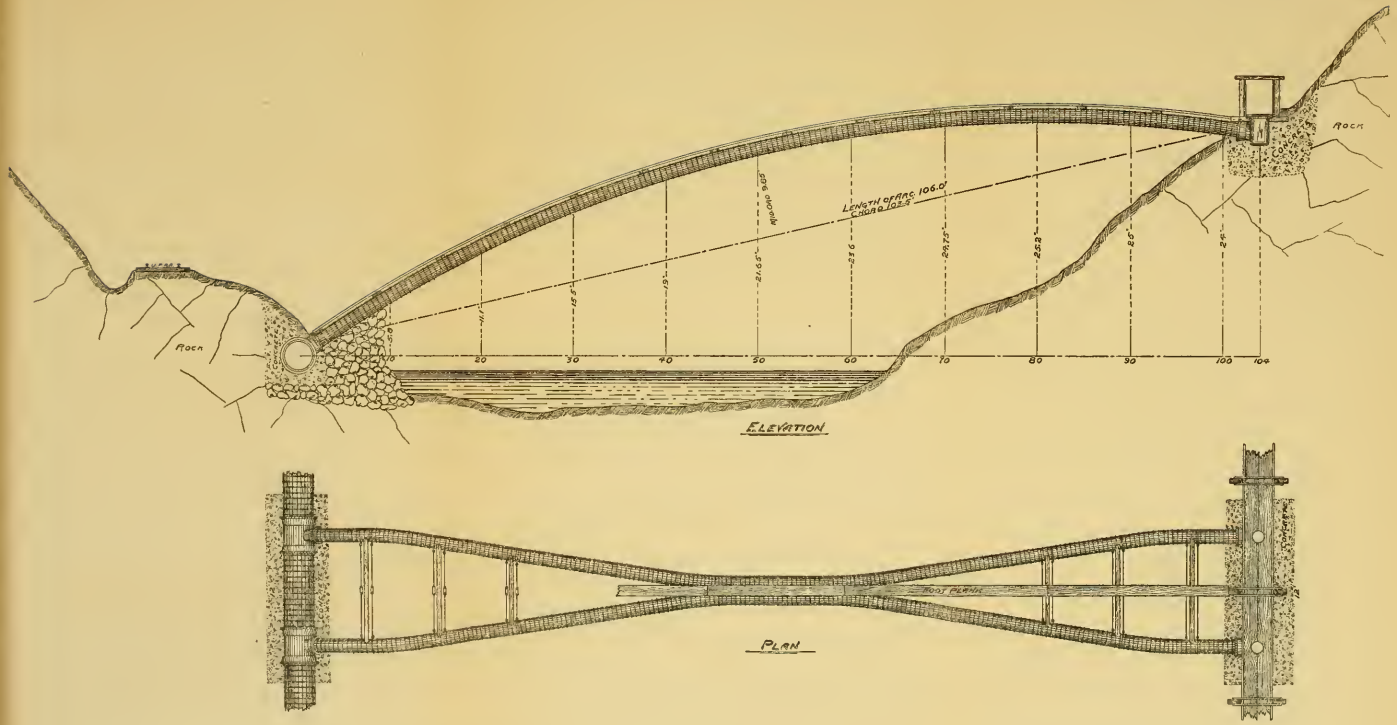
combined capacity of 30 000 000 galls. daily. The elevation of the reservoir at high water line is 5 545 ft. above sea level, or 350 ft. above city base, which was too high to permit its being fed from the river through the 34-in. conduit; and a third and larger conduit is to be built to supply the lake. This will necessarily start 1½ miles higher up the cañon than the originally located head of Conduit No. 2, and it was decided to make but one diverting dam for both conduits, and make but one conduit large enough for both lines through the principal part of the cañon work, or to a point known as Tunnel 1.

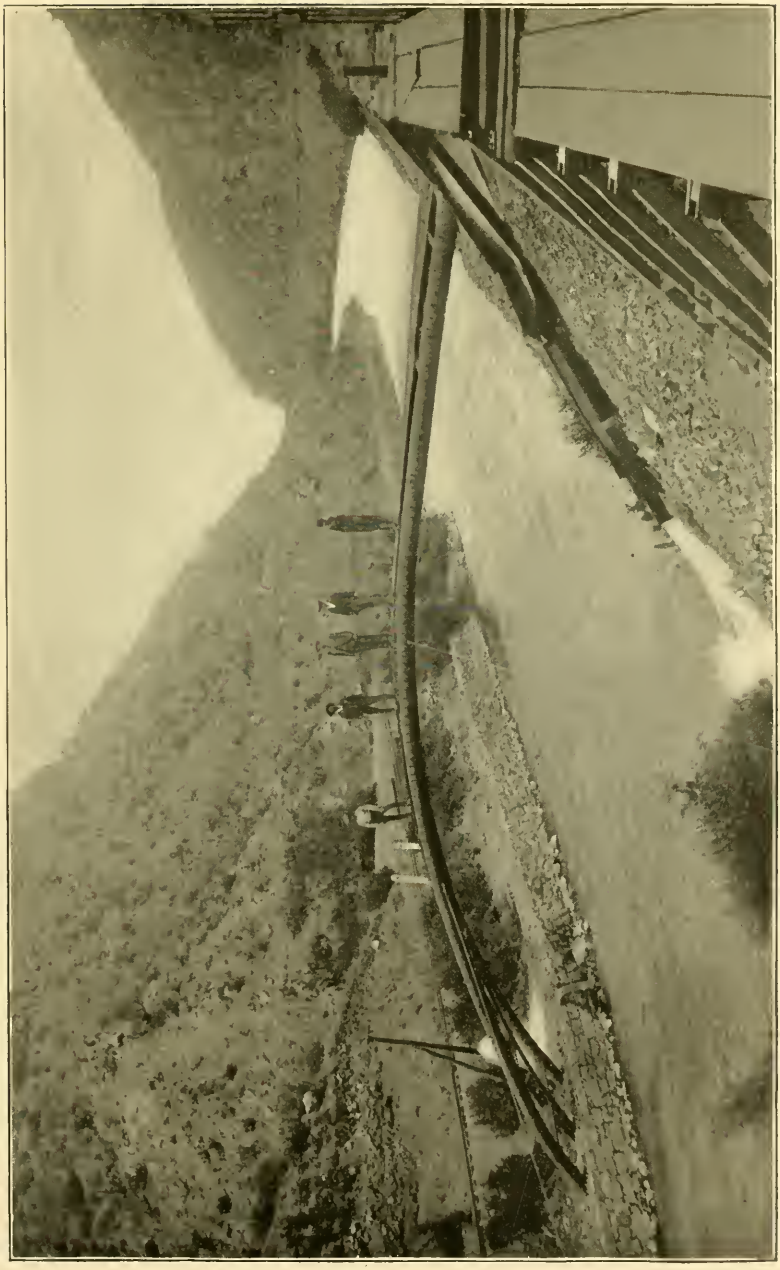
The location for the filters for the second conduit was selected on property owned by the company some four miles below Tunnel 1. Between the filters and the city the hydraulic grade was established for the desired capacity, but between Tunnel 1 and the filters the difference in level was sufficient to give a capacity of 18 000 000 galls. daily to a 30-in. pipe, thus giving an ample surplus over the capacity of the 34-in. line, for use in washing the filters. Accordingly, the diameter was reduced to 30 ins. above the filter plant.

Tunnel 1 cuts through a sharp granite ridge, which is also pierced by the High Line Canal Company, whose tunnel is 25 ft. lower and about 100 ft. distant at the north end. Pending the completion of Tunnel 1, and the dam and aqueduct above, a temporary arrangement was made with the High Line Canal Company to use their tunnel and take water at a waste gate at its mouth into the 30-in. pipe. A short flume was built from the waste gate, with sand gates and regulators, and connected with the 30-in. main on the opposite side of the river by means of two 13-in. wooden pipes built in the form of an arch spanning the stream. The entire span from the center of flume to the center of the 30-in. pipe is 104 ft. These pipes are spread apart 12 ft. at the ends and are brought together at the center for a distance of 20 ft. where they are tied together with iron bands. This construction, which was made as a cheap and temporary crossing, is novel, but is stanch and effective, and illustrates what may be done with wood pipe. Plate X shows the plan of the structure, from which a slight deviation was made owing to the fact that the pipes refused to bend exactly to the lines of the drawing, but assumed curves of their own. Plate XI is a side view looking down the river.

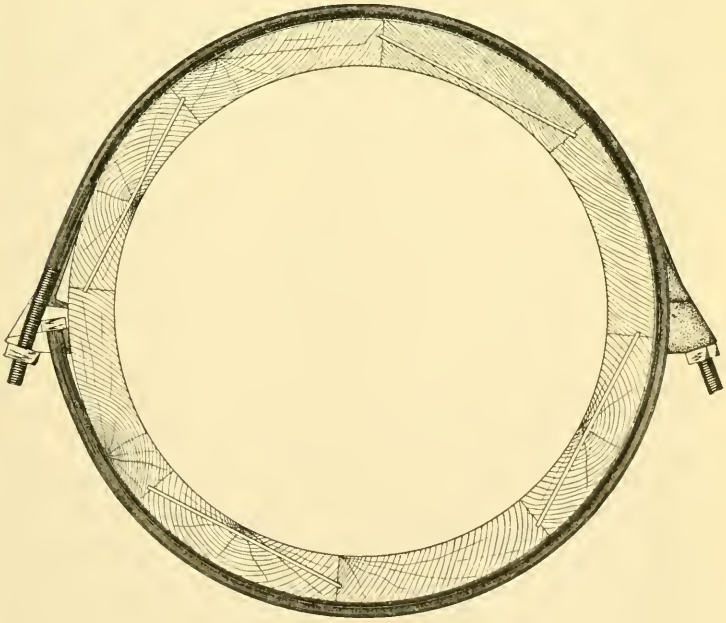
The arch was built on light false work and was finished in less than a week from the time the materials were assembled. When the false

PLATE X.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 691.
SCHUYLER ON WATER WORKS OF DENVER.





work was removed and the pipes were filled with water the maximum deflection was but $\frac{7}{8}$ in. with eight men standing on it. It is constantly used as a footbridge, and has but little vibration, even in a gale of wind. The bands around the pipe are $\frac{3}{8}$ in. diameter, spaced evenly $2\frac{1}{2}$ ins. apart. The lumber used is Texas pine, $1\frac{1}{4}$ ins. thick, in 12-ft. lengths. In this case the end joints were made every 6 ft., each alternate stave extending to the next joint. The special castings at each end were set in a heavy bed of concrete, filling the space between them and the solid rock in place, the whole interwoven with small steel rods. The arch has no tension member and one end is 24.5 ft. higher than the other, which is another unusual feature, although one which cannot be said to add to its value.



A somewhat similar arch occurs at another point on the line where the 34-in. pipe crosses a canal and is unsupported for about 60 ft. In this case, the versed sine is about 3 ft. As it was made without a drawn plan no definite data can be given of it, but it is simply mentioned as a further illustration of the ease with which short spans can be

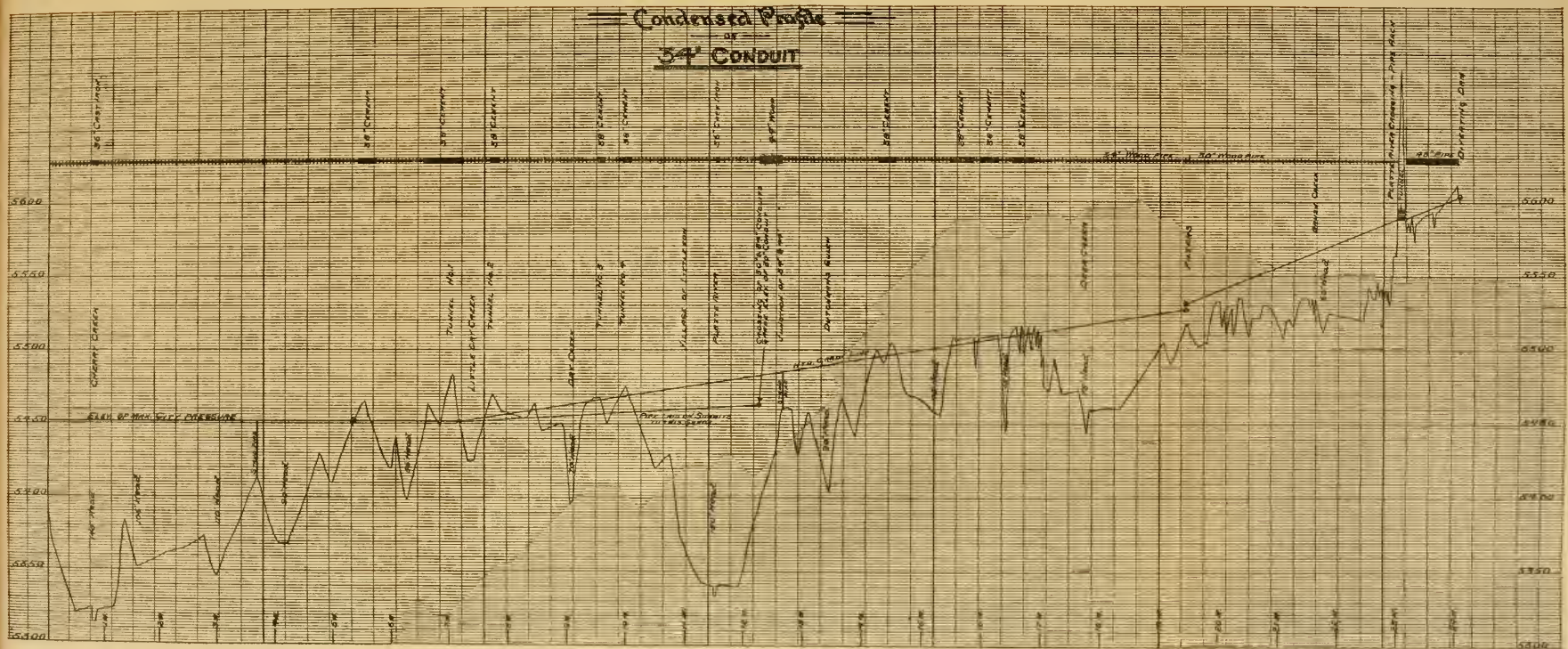
made, it having been constructed precisely like all the rest of the pipe.

A condensed profile of the second conduit is presented in Plate XII. At all the high points where the hydraulic grade is below the ground surface, a pipe of Portland cement concrete was constructed, either in open trench or tunnel. The remainder of the line is of wood pipe, except the crossings of Platte River and Cherry Creek, where 36-in. cast-iron pipe was used. California redwood was employed for the greater portion of the wood pipe, and a part was built of long-leaf yellow pine from Texas.

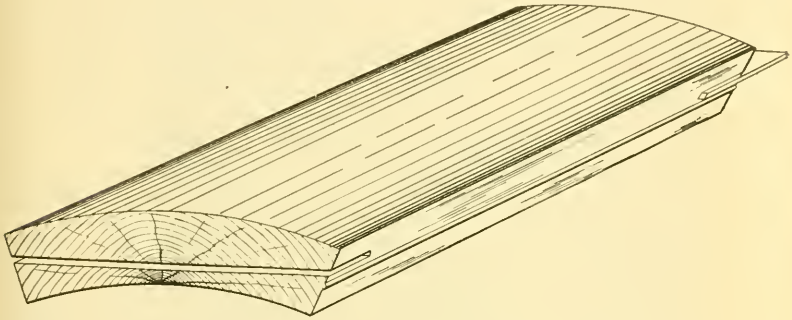
The fact that the entire water supply of a city of 150 000 inhabitants is conveyed in wooden mains of this type is so radical a departure from all precedents that it is deserving of more than passing notice. It is manifestly and unreservedly successful, and has achieved an enormous saving in cost; and if successful and economical here, there is no reason why it should not be equally so in other localities, nor why its merits should not meet with general recognition, freed from preconceived notions of its alleged temporary nature and lack of durability.

One of the strong features of the wooden stave pipe is that it is built continuously in the trench where it is to lie, and not in sections made in the shop; hence, it involves no difficulties in handling, transportation, loading and unloading, lowering into the trench, etc., as with other classes of pipe, which become serious in the ratio of their diameter. A defect in a stave or a band or a shoe does not involve the rejection of a section. Being made continuous, of staves of irregular lengths, it has no defined end joints, although the sections are generally joined within a space of 2 or 3 ft., over which space it is customary to distribute three extra bands wherever the spacing is less than 150 bands per 100 ft. The butt joints are "staggered," so that no two are nearer each other than 6 ins., and from that to 3 ft. The staves are dressed in the planer to conform to the true curve of the pipe, both inside and out, and hence are of uniform thickness and must be evenly saturated when the pipe is filled and in use. To insure saturation from the inside, they are made as thin as possible consistent with strength, rigidity and ability to hold up under a load of cover. The edges are dressed to exact radial lines, with a minute tongue or calking bead $\frac{1}{16}$ in. high on one edge of each stave. When the pipe

PLATE XI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 691.
SCHUYLER ON WATER WORKS OF DENVER.



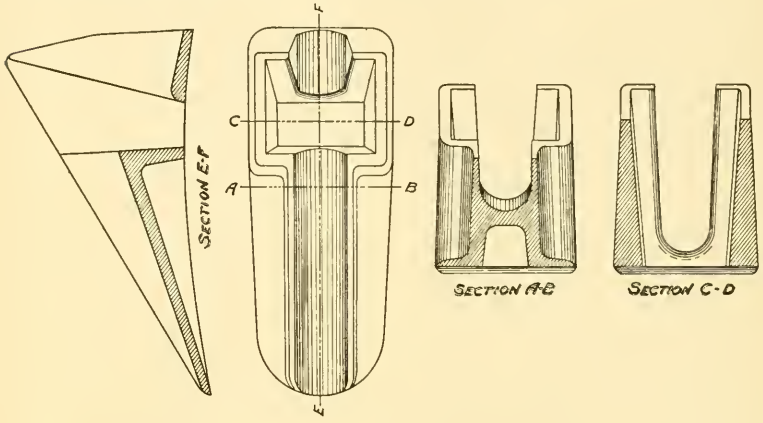
is tightly drawn together, this is embedded in the adjoining stave, and serves to take up any slight irregularity in the planing of the edges. Leaks seldom or never occur along the longitudinal seams.



Metallic tongues are inserted in the ends of the abutting staves $\frac{3}{4}$ in. deep by means of a saw kerf. These tongues are cut $\frac{1}{4}$ in. longer than the width of the stave, and thus project $\frac{1}{8}$ in. on each side, the projection being embedded into the wood of the adjoining stave when the bands are tightened. This serves to cut off leakage around the ends, which would otherwise occur whenever the corners of the staves were slightly jammed or shrunken. The end slots to receive the tongues must be cut with precision, both as to depth and position, to avoid irregularity of surface of the pipe, both inside and out.

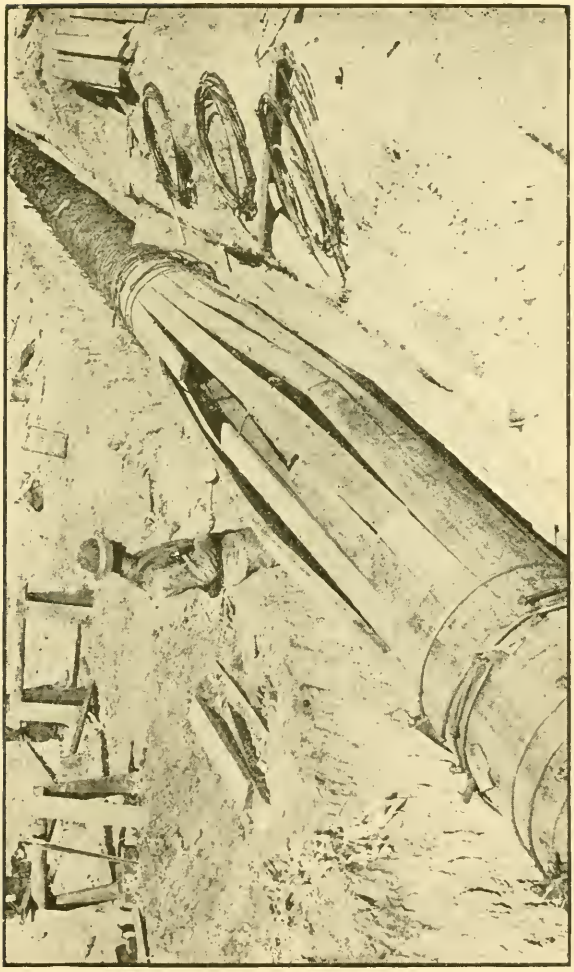
The bands used are made of mild steel, having a tensile strength of 58 000 to 65 000 lbs. per square inch, and an elastic limit of 40 000 to 50 000, with an elongation of 24 to 27% in 8 ins. They are generally made 4 to 5 ins. longer than the exterior periphery of the pipe, it being desirable to have considerable spare length to facilitate the rapidity of the work of erection, as too much time would be lost in "trying to make both ends meet," if the bands were barely the right length. They are made with a head on one end, and nut and thread on the other, the threads being cut 5 ins. It has been found advantageous to have the threaded ends upset so as to make the bands of equal strength throughout, as recently improved processes render it feasible to upset small rods for a length of 5 ins. The ends of the bands are brought together with a cast or malleable iron shoe of peculiar construction having a slot near the bottom to receive the head end of the band, the other end dropping in over the head between the two jaws, against

which the nut and washer set, producing a straight and direct tension on the band. Malleable iron shoes are preferable to cast iron, as the latter may be easily broken by a careless blow when under tension; and as the malleable shoes can be made very much lighter, their cost is little, if any, greater. Malleable shoes were exclusively used on the second conduit. Their weight is but $1\frac{1}{3}$ lbs. each. One pattern was used interchangeably on the 30, 34 and 44-in. pipe.



Beginning at the bottom, the staves are set up inside a U-shaped mould, usually a piece of $1\frac{1}{2}$-in. gas pipe. When the half diameter is reached, an inside form, completing the circle, also of gas pipe, with the ends lapping and spread apart so as to stand alone, is placed inside the half pipe, and the requisite number of staves added to complete the full pipe. The foreman is furnished with a profile on which the number of bands for each station is marked, and he marks on the top of the pipe the exact position of each band, spacing them according to the profile. Then the bands, which have previously been bent around circular bending tables of proper diameter and coated with paint or dipped in a hot bath of asphalt, are slipped on over the ends of the staves, distributed to position, the shoes attached, and the nuts partially screwed up. While the bands are being adjusted, lined up perpendicular to the pipe and partially tightened, the pipe is rounded out and "coopered," to bring all the staves out full, and the staves are carefully driven home to meet the abutting staves. The driving of the staves is repeatedly gone over as the bands are tight-

PLATE XIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 691.
SCHUYLER ON WATER WORKS OF DENVER.



ened. The final "cinching" of the pipe before it is covered with earth is usually deferred till the day after it is laid, when every nut is gone over and brought up to extreme tension. Brace wrenches are used with levers not over 10 ins. long, as a precaution against over cinching. It is desirable to draw the bands tight enough to imbed them slightly into the wood, but where they are but 2 or 3 ins. apart great care must be taken to avoid crushing the wood or collapsing the pipe. This frequently occurs, and has been the cause of the most serious leaks when the pressure came to be applied. If the staves are quarter sawed and the grain runs obliquely across, the excessive tightening is apt to cause a split and a lapping of the two parts, which might escape detection until the pressure was on.

A pipe-laying gang consists of from 8 to 16 men, according to the number of bands per unit of length. One-half the gang is usually employed in back-cinching. The speed of laying is dependent upon the number of bands to be placed, as well as upon the diameter. On the 34-in. pipe the gangs usually placed 700 to 1 000 bands per day, laying from 150 to 300 ft. of pipe. On the 44-in. pipe the rate was about 500 bands per day. The cost of erection is from 5 to 10 cents per band, on pipe from 30 to 48 ins. diameter.

It is customary to distribute the gangs 1 000 to 2 000 ft. apart, and where they meet the coupling is made with ease, the last staves being cut $\frac{1}{8}$ in. long and sprung into position; thus the buckle joint (see Plate XIII) is the tightest of all. This facility of construction renders it possible to lay a line very rapidly when the trench is ready and the materials are distributed.

To make good pipe it is essential that only the best of lumber be used—clear, straight-grained, without knots, or checks, or season cracks, or flaws of any kind. It should also be thoroughly dry and seasoned before milling. Then, after it is milled, the sooner it is built into the pipe the better. If milled when only partially seasoned it is apt to shrink unequally in width, and more at the ends than at the center. This end shrinkage causes troublesome joint leaks. The shrinkage in California redwood, that had seasoned 60 to 90 days before milling, was frequently as much as 3 ins. in the 20 staves that formed a 34-in. pipe. This had to be met by the insertion of a narrow stave of varying width to fill the full diameter, as the bands are of an exact length and cannot be made to accommodate the lumber.

The formula used for the distribution of the bands was :

$$N = \frac{1\,200\ dp}{2s}$$

where N = number of bands per 100 ft.,

d = diameter of pipe in inches,

p = pressure in pounds per square inch,

s = safe working strain in pounds, for bands when threaded for use, determined by regular tests at the mills where they were made.

In making up the tables used for the distribution of the bands, values of s were taken as follows :

¾-in. bands,	plain	s	=	1 000
⅜ " "	upset	s	=	1 200
½ " "	plain	s	=	2 000
½ " "	upset	s	=	2 500
⅝ " "	plain	s	=	3 000
⅝ " "	upset	s	=	3 500

These values of s give a factor of safety of about 5 in each case, or about one-fourth of the elastic limit.

Changes in the rate of spacing are made with every 5 ft. change in piezometric head, with the exception that for low heads the limit of distance from band to band is fixed at 17 ins.

All the high-pressure pipe in use that has been examined by the writer was found in a condition of complete saturation, with moisture oozing slightly over its entire surface. For perfect preservation from decay this seems to be the ideal condition. Numerous samples have been taken from pipe in use from three to nine years in this vicinity, and in every instance it has been found perfectly sound. Hence it is fair to assume that the wood of the pipe is likely to be as long lived as cast iron, so long as it is kept in this condition by constant use. The bands are the only perishable portion of the pipe, and these, when properly coated, should be good in ordinary soils for 15 to 20 years.

Bands that have been in place nine years do not yet manifest any serious corrosion. Moreover, they may be easily replaced without interrupting the use of the pipe, and it is a simple matter to replace a defective stave here and there.

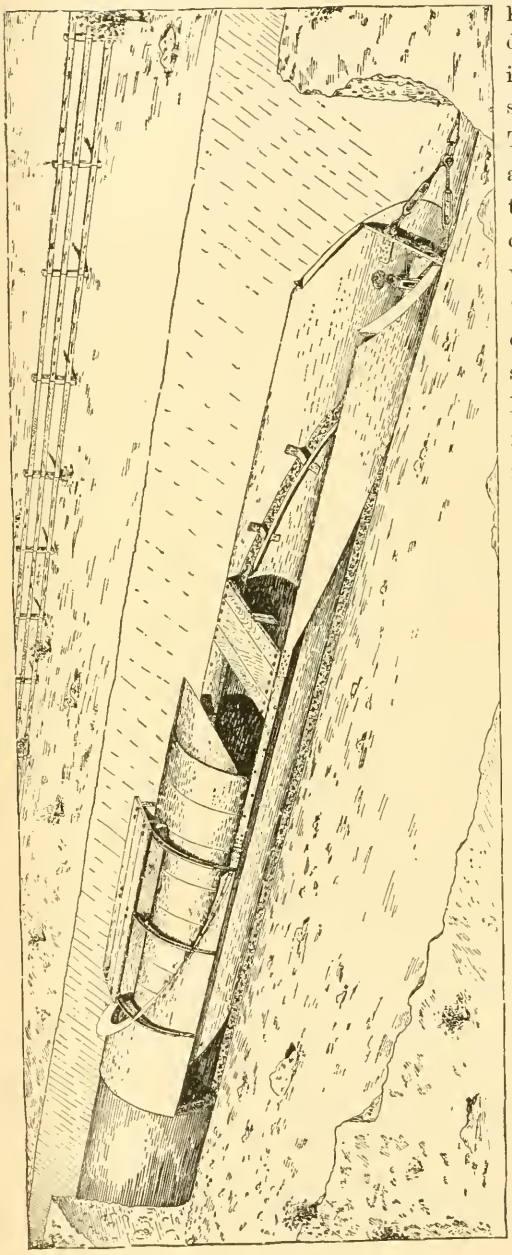
One advantage possessed by the pipe is quite evident, especially when open at the lower end; it can never burst, as cast iron frequently does, without apparent cause or warning, and sometimes with disastrous damage to property. It may spring a leak, but it gives ample warning of any serious weakness. Leaks are sometimes allowed to run for months, and they never increase materially and give no anxiety. A few pine wedges are usually all they require, and sometimes a toothpick will serve.

If the pipe can be allowed to fill very slowly and soak for weeks under little or no pressure before the full head is turned on, there will be very few leaks developed subsequently, if the work of erection has been properly done and good materials used. Extreme care and inspection of every stave, seam and joint in construction is well repaid when the pipe finally comes into use.

Outlets for laterals of over 6 ins. diameter are usually made by means of cast-iron tees, through which the wood is built, the opening of proper size being sawed out afterwards. The bell ends are made about 5 ins. deep, with a space of ¾ in. for oakum calking. For small connections a cast-iron saddle is strapped around the pipe, with lead or rubber gasket underneath. For ordinary house services, from ½ to 2 ins. diameter, a hole is bored into the wood and the service cock screwed in.

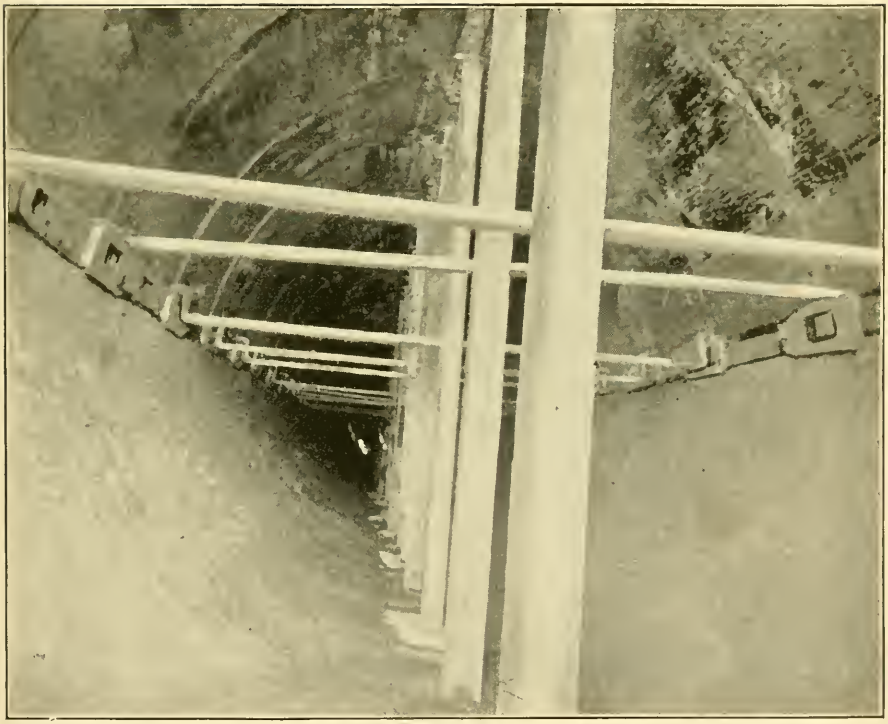
At a moderate estimate the saving effected by The Citizens' Water Company by the use of wooden pipe for their main conduits has been no less than $1 100 000 over the cost of cast-iron pipes of equal capacity. The interest on this amount at 6% would renew the mains every five or six years, or duplicate them as often as that if necessary. With such a showing as this, the correctness of which cannot be challenged, there can be no doubt of the soundness and wisdom of the judgment which governed the selection of wood as the material for the main conduits of the Company. If this be admitted, why may not the experience of this case be repeated anywhere, with equally favorable results?

Cement Concrete Pipe.—As the line of the second conduit follows generally the country roads, little choice could be made to secure uniformity of depth of cutting. Where the grade line was deeper than 14 ft. below, the high points were tunneled and lined with cement concrete 3 to 4 ins. thick. Where the depth was from 6 to 14 ft., an open trench was excavated and concrete pipe formed in the bottom. A large



portion of this work was done with a pipe mould invented by E. L. Ransome, of San Francisco. The machine consists of a wrought-iron shell of the size of the inner diameter of the pipe, to which was attached a "leader" and "saddle" of larger diameter, the space between which left a ring or core, that formed the pipe when filled with concrete. The mould was drawn along the trench slowly by a cable and horse-whim, and, as it moved, the concrete was shoveled and rammed into the core space on a long incline that gave room for tamping it throughout its circumference. The top half of the pipe was supported by sheet-iron plates, 2 ft. wide, that were placed one after another on the forward end of the mould, and, being clamped firmly together at the top and sides, remained in position while the mould slid out from under them. They were supported,

PLATE XIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 691.
SCHUYLER ON WATER WORKS OF DENVER.



after leaving the mould, by sticks placed under the center of each, and held to an uniform width by small clamping rods (see Plate XIV). The plates were allowed to remain in from 24 to 48 hours, until the concrete had set sufficiently to be self-supporting.

A gang of 30 men operated the machine and performed the various functions of mixing, wheeling, shoveling and tamping the concrete, extracting the plates in the rear, bringing them forward, cleaning and greasing them, and putting them in position. When everything was in good working order and the concrete could be mixed and delivered without delay, the machine was capable of making about 600 ft. of pipe per day, and on short runs would go at the rate of 900 ft. per day; but counting all stoppages, the rate did not exceed 300 ft. per day. The inner diameter was 38 ins., and the thickness of shell $2\frac{1}{2}$ to 3 ins. The bottom was moulded flat for 18 ins. The concrete was mixed in a mill, also invented by Mr. Ransome, which automatically measured the cement, sand and gravel in the desired proportions, and delivered the concrete, thoroughly mixed, in a continuous stream at the rear, whence it was shoveled into wheelbarrows. It was mounted on a 4-wheeled truck, with a small engine to operate it, and moved as often as necessary to keep up with the work. The superiority of the quality of concrete mixed by machine over that mixed by hand was apparent at a glance, when the two were compared in finished work, although no experimental tests were made to determine the relative strength of each. The proportions used were $3\frac{1}{2}$ of river sand and gravel to 1 of cement.

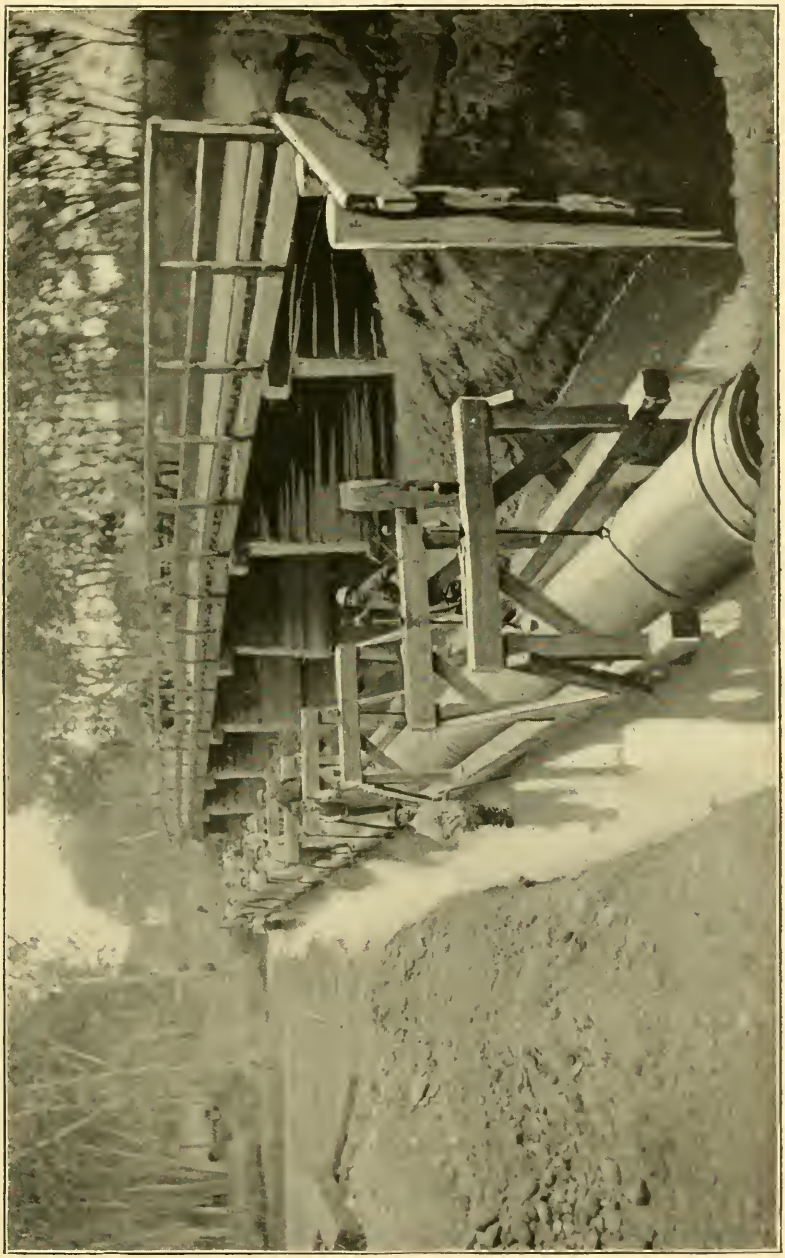
The total length of concrete pipe laid on the line of the second conduit was 14 470 ft. in nine separate sections of 600 to 2 200 ft., of which five were tunnels with a total length of 2 730 ft. The amount constructed with the machine was 8 900 ft., including one tunnel 700 ft. long. In the machine-made pipe, the weight of the machine and the gang of men riding on it aids greatly in compacting the concrete. The smooth surface of the inner mould gives a neat finish to the bottom half inside, while the saddle dragging over the top trowels down the outer surface on top. The pipe was washed with pure cement applied with brushes after the removal of the plates.

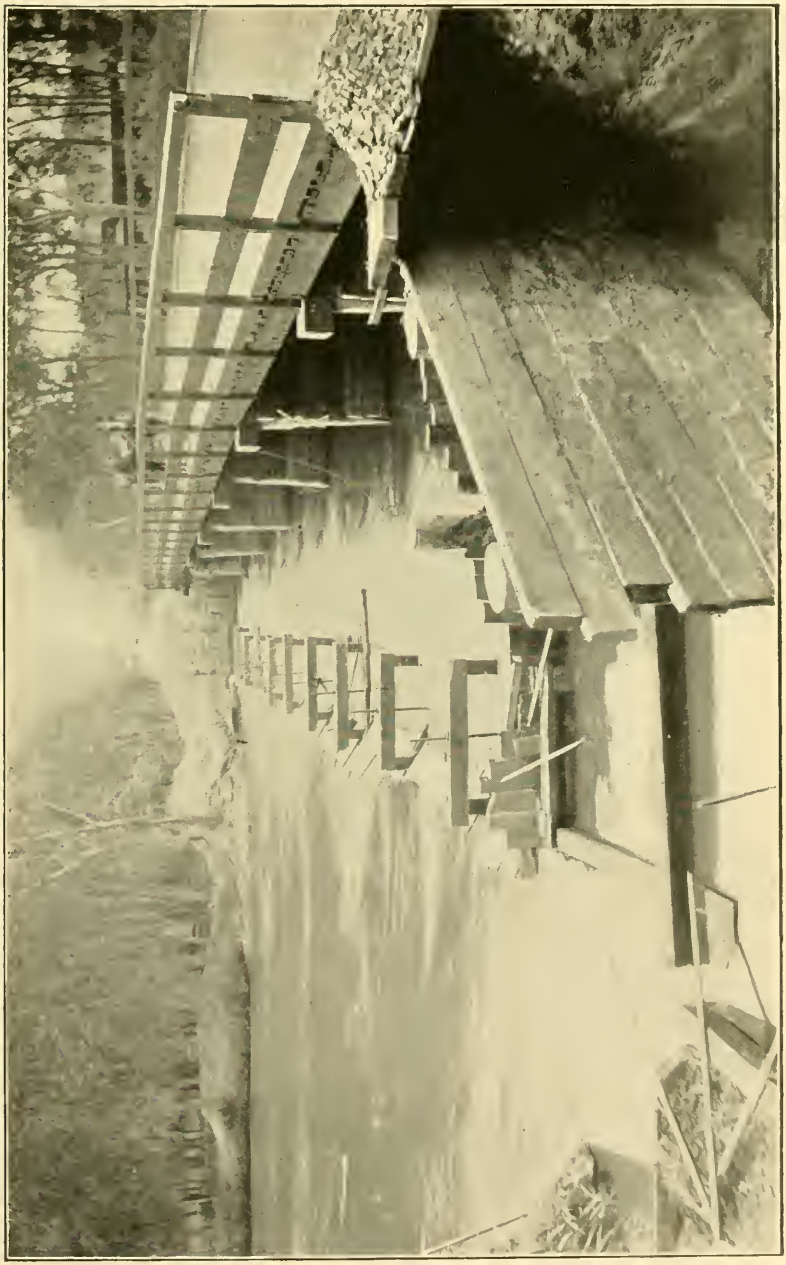
The cost of the machine-made pipe was $1 35 to $1 50 per foot, with cement at $3 75 per barrel; gravel, $1 25 per cubic yard, and labor, $1 75 to $2 00 per day. This was after the men were broken to the routine of their respective duties, and everything working smoothly.

The hand-made pipe was moulded on wooden sectional forms, 3 ft. long, the bottom in one piece reaching to within 3 ins. of the center on each side, while the top form, hinged in the center so as to fold inward, completed the circle. The thickness was nowhere less than 3 ins., and generally 3½ to 4 ins. In the tunnels, the entire space between the mould and the solid walls was filled with concrete, giving a thickness of 4 to 6 ins. all around. The contract-price for excavation of tunnels was $1 25 per linear foot. Shafts were sunk at intervals of 150 ft. The lining followed the excavation as quickly as possible, and before disintegration and crumbling of the earth took place.

One section of the concrete pipe was laid over a succession of short fills and cuts, the maximum depth of fills being 13 ft. They were near the crest of a ridge, and none of the little gulches have water-sheds over ¼ acre in extent, so that no culverts through the banks were needed. The fills were made of coarse gravel, sand and clay, and packed firmly. To support the pipe from possible settlement, posts or piers of concrete were put down through the embankment to the solid ground beneath by boring holes with an 8-in. post auger and filling them with concrete, in which were placed twisted steel rods ¾ in. square. These piers were put in rows 2 ft. apart, and over them was laid a concrete floor 4 ins. thick, 2 ft. wide, also containing twisted rods laid in longitudinally near the bottom, to take the tension, with cross rods every 4 ft. The pipe was built up on the floor. The embankment was carried up on each side of the pipe and high enough to cover it 2 ft. in depth.

The sinking of the cast-iron pipe under the beds of the Platte River and Cherry Creek were interesting features of the construction of the second conduit. The former was 156 ft. long, the latter 350 ft. In the Platte River crossing, the stream was forced to scour out the trench under the pipe as it was lowered in one section, on screws between rows of piles (see Plate XV). Sixteen long screws suspended the pipe and held it precisely level, and, with a man at each screw, it was lowered with exact uniformity. The bed of the river consisted of coarse gravel, and there were frequent hard spots or reefs that required digging, to assist the action of the current. The shore ends had to be dug down by hand entirely, but the greater portion of the work was done by the stream. When the pipe was down, so that the water began to run over the top, a dam was formed of boards set on an incline, resting





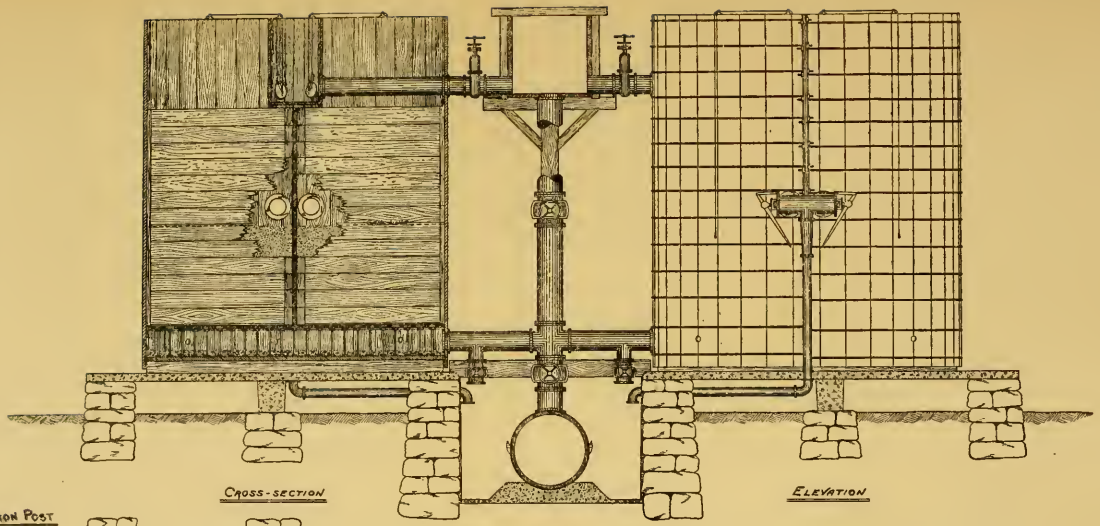
loosely against the pipe and the stringers on the piles (see Plate XVI). In this way the pipe was lowered to a depth of 2 ft. below the lowest part of the channel.

Another method was employed at the Cherry Creek crossing, where the stream bed is entirely of sand, with but little water flowing. A row of piles, 20 ft. long, was driven flush with the surface on the lower side of the pipe for it to rest against, and, when the joints were run and calked, it was lowered without suspending simply by digging the sand from under it, a little at a time, care being taken to excavate uniformly. To keep the trench free from water, a drain ditch was scraped out of the sand for a distance of about 1 000 ft. down stream. When the pipe had reached a depth of 4½ to 5 ft. beneath the surface, the joints were recalked and the trench allowed to refill, which it did at the first little freshet.

The two conduits cross each other at an angle of 61°, and here a somewhat novel connection was made, by means of which water from each of the mains can be turned into the other, or both be used independently. There are four valves in the cross, one in each of the main lines, and one in each of the by-passes. The entire cross, with its valves, was fitted together on the surface and the joints made with lead and calked, and the whole lowered in one piece with screws. Its weight was about 30 tons, and it rests in a bed of concrete that envelops it. Immediately below the cross Venturi meters are to be placed, to give a measure of the total volume of flow into the city.

The Filter Plants.—When the plan for the second conduit was adopted, and the mechanical filtration of its water supply was decided upon as a resultant necessity for at least a portion of the year, an investigation of the cost of the mechanical filters on the market led to a research for some cheaper way of accomplishing the same results on a large scale. The plans finally adopted, which are the invention of Mr. C. P. Allen, chief engineer of the company, bid fair to be, not only thoroughly successful, but in many respects an improvement upon the well-known types of filters in use, particularly in the matter of rewashing and cleansing the filtering material, while the cost of the plant per unit of capacity is but one-fifth to one-fourth the cost of efficient filters on the market. Plate XVII illustrates the general details of construction of the filters, which consist of a series of wooden tanks, 12 ft. in diameter, 14 ft. high, ranged in rows on either side of

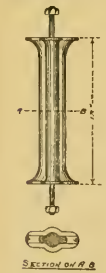
the main pipe. These tanks have a false floor of wood, 1 ft. above the bottom, supported and firmly attached to the bottom by cast-iron posts 1 ft. apart, with bolts running through both floor and bottom. In this false floor strainers of finely perforated brass plate are placed in holes bored through the floor 6 ins. apart, there being 500 strainers 2 ins. in diameter in each tank. The tank above the floor is divided into four compartments by wooden partitions reaching to within 4 ft. of the top. Clean quartz sand to the depth of 3 ft. is placed in each of the compartments. This is dried, screened and crushed, only that which passes a No. 20 screen and is retained on a No. 30 screen being used. The particles are therefore between $\frac{1}{40}$ and $\frac{1}{50}$ in. in diameter. Each compartment is provided with a valve for washing out the sedimentary deposit of filtration, this valve being placed about 18 ins. above the top of the sand. A valve in the main pipe, when closed, forces the water to rise into a flume passing the whole length of the plant at the level of the top of the tanks. Outlets with regulating valves connect the flume with each of the compartments of each tank. The filtration is downward, the clear water being collected between the floors of the tanks, whence it is conducted again into the main pipe below the main shut-off valve and continues on its way to the city. The washing of the filter beds is accomplished by a reversal of the current, and either filtered or unfiltered water may be used for washing. To use the filtered water for the cleansing process, it is necessary to throttle off a part of the clean water flowing from the bottom chamber to the main by partially closing the valve that controls it; then, by opening the waste valve to any one compartment, the water in that compartment is drawn down several feet lower than in the other compartments, and an upward current under pressure takes place through the sand from the chamber below, which is common to all the compartments. When the waste water runs clear, the waste valve is closed and that of another compartment opened, until all have been successively washed. The capacity of each tank is expected to be about 500 000 galls. in 24 hours at the minimum. This will require a velocity of about 5 ins. per minute through the sand, which is much greater than would be desirable or efficient in ordinary open-pond filter beds, but with the aid of a coagulent to ensnare and precipitate the particles of sediment, and by frequent washing—once or more daily—the rate given is not believed to be excessive.



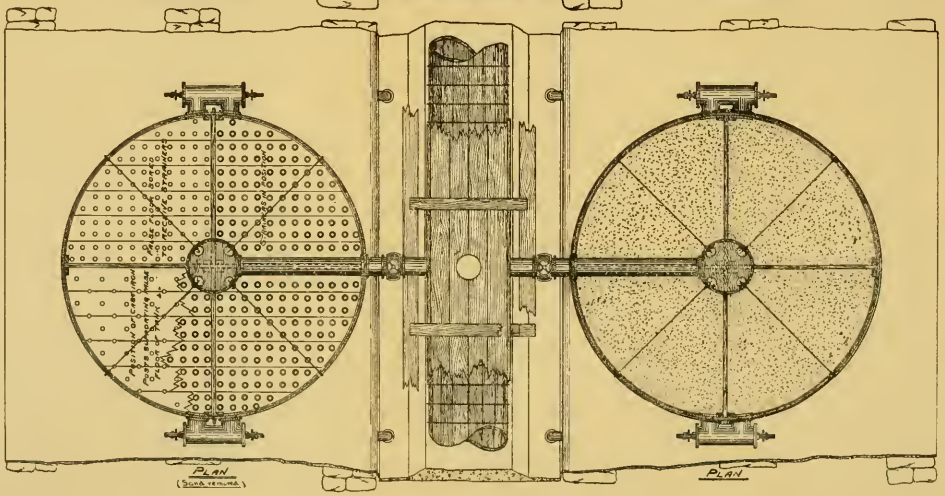
CROSS-SECTION

ELEVATION

CAST-IRON POST



SECTION ON A B



PLAN
(TOP REMOVED)

PLAN

STRAINER

PLAN

SECTION ON A B

The plant has not been in operation a sufficient length of time to determine fully the duty of each tank on water of different grades of impurity, nor the cost of filtering the water, but it is estimated not to exceed $5 per 1 000 000 galls. at any time.

The foundations of the tanks consist of parallel lines of masonry walls, over which a 4-in. concrete floor is laid, with a grillage of ¾-in. square steel rods, 2 ins. apart in one direction, 3 ft. apart in the other, embedded in the concrete ¾ in. from the bottom.

At the main outlet to Marston Lake reservoir a similar filter plant has been erected, consisting of a battery of 10 tanks. The water of the lake, being comparatively free from sediment, will require but little effort or expense to render it clear, pure and sparkling.

A settling flume is in contemplation at the upper end of Tunnel 1, on the second conduit, to precipitate all sediment that will not be transported with a velocity of about 0.2 ft. per second. This will clarify the water to a considerable degree and increase the duty of the filters, as the mountain water settles quickly on being brought to rest, and seldom displays the milky cloudiness that is more troublesome to get rid of.

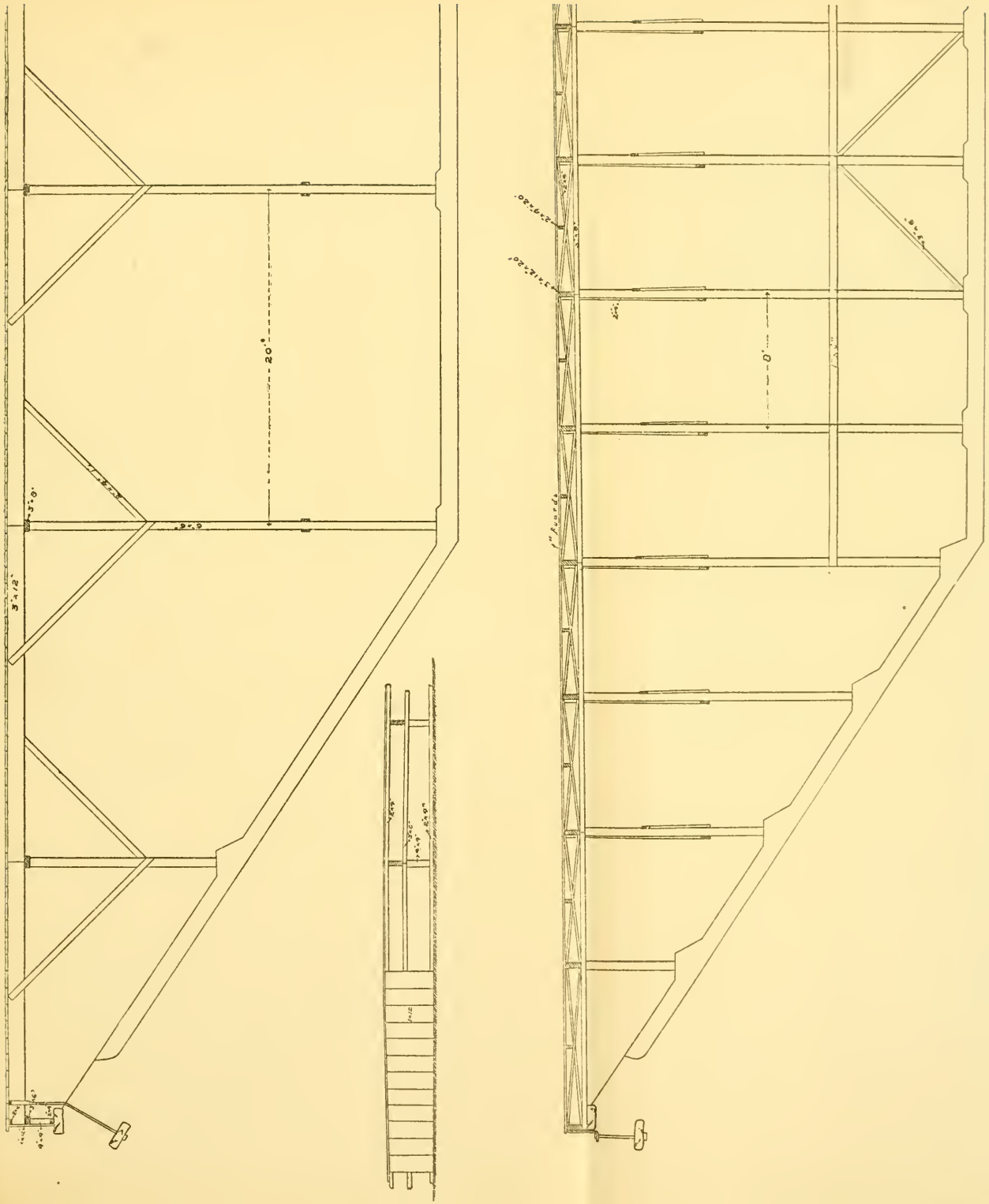
Reservoirs.—The first conduit ends in two distributing reservoirs on Ashland Avenue, some four miles west from the center of the city to which reference has already been made. A description of their asphaltum lining was given in Vol. XXVII, page 629, of the *Transactions* of the American Society of Civil Engineers. The reservoirs were covered with flat roofs of rough pine boards, supported by wooden posts 6 x 6 ins., placed in rows 20 ft. apart, and spaced 10 ft. apart in the rows (see Plate XVIII). The cover was not made tight, but was laid with spaces of 1 in. between the boards, which saved 10% in the lumber, without impairing the efficiency of the roof as a preventive of the growth of algæ, which was its sole object. It is perhaps needless to say that this purpose has been effectually accomplished. Panels at convenient intervals were arranged in the roof, to be lifted out when desired, affording openings for shoveling in snow whenever its depth on the roof became objectionable, as well as to give light, ventilation and easy access to all points in the reservoir when it is being cleaned. To facilitate a circulation of the water in the reservoir and maintain a constant movement in the mass of water, the pipe which serves as both inlet and outlet was extended, in the case of the east reservoir, around

two sides in the form of a square flume or box, tapering from 3 x 3 ft. to 1 x 1 ft. with 6-in. openings at intervals of about 50 ft. This was firmly fastened to the bottom, to prevent floating. With this device no portion of the water becomes stagnant, but is always kept in slow motion, whether the current be in or out.

On the opposite side of the city, near the terminus of the second conduit, two iron tanks have been erected on the top of an embankment 20 ft. high, the tops of which are level with the high water-line in the Ashland Avenue reservoirs. They are each 50 ft. in diameter, 45 ft. high, and have a combined capacity of 1 320 000 galls. They serve a useful purpose as equalizers to maintain supply and pressure to that part of the city, and to relieve the second conduit main pipe. They rest on a concrete base 2½ ft. thick, tied together with twisted steel rods.

Marston Lake Reservoir.—The main storage reservoir of the system is a natural basin near the foot of the mountains, known as Marston Lake. This basin, without any artificial embankments, has a capacity of 2 272 000 000 galls., covering 383.5 acres, and by the erection of dikes across two low gaps in the surrounding rim of the basin its capacity can be enormously increased; but a limit was fixed at 5 637 500 000 galls. as the ultimate capacity desired, which brought the high water-line 20 ft. above the lowest points on the gaps referred to. One of the dikes will be 3 300 ft. in length, the other, 2 100 ft., at a height of 5 ft. above the water-line. The material available is suitable for these embankments and can all be taken from the interior of the basin. The reservoir, when full, will cover an area of 647 acres and have a maximum depth of 67 ft.

The main outlet to the reservoir, the one connecting with the two conduits already built by the 44-in. pipe heretofore mentioned, is at the southeast corner of the lake, and consists of a tunnel 1 600 ft. long, driven to tap the lake at its lowest bottom. It is 38 ins. in diameter and is lined with concrete 5 to 6 ins. in thickness. The greater portion of its length is in yellow shale and soft sandstone. Flanges 12 to 16 ins. deep, 12 ins. wide, were cut entirely around the periphery of the tunnel at intervals of 50 ft. and filled with concrete during the process of lining, to interrupt percolation between the lining and the natural walls of the tunnel, although, considering the very firm nature of the material and the thoroughness with which the concrete was rammed into



every space, this was, perhaps, a needless precaution. A valve well or shaft was sunk in the summit of the rim of the basin, 900 ft. from the inner portal of the tunnel, to intersect the tunnel. It is 12 ft. in diameter, lined with stone masonry 2 ft. thick, and was carried up above the surface to the level of the top of the dike when finished. The dike will be built around this shaft. A second tunnel, 25 ft. above the lower one, and of the same size, was run from the interior as far as the gate shaft, 600 ft. This tunnel is expected to be the one principally used, as it has nearly 90% of the total reservoir capacity above it and will draw water nearer the surface, where the quality is generally better than at the bottom. Moreover, the water from this tunnel, on reaching the valve shaft, will have a vertical drop of 25 ft. to the level of the outlet, and will thus be well aerated. The shaft is carried down 10 ft. below the outlet tunnel, forming a sump to receive the shock of the falling water. Gratings of iron across the shaft will also serve to break the water into spray and mingle it more thoroughly with air. Pressure valves, to be operated from the top of the shaft, are placed at each of the tunnels on the lake side, and a double set of perforated plate screens extend from top to bottom across the shaft.

The inlet valves, at the inner portals of the tunnels, are novel in their construction and operation (see Plate XIX). A railroad track, with 30-lb. T rails, and cast-iron cross-ties resting at each end on concrete piers 12 to 18 ins. above the ground, is laid from the high-water level down the slope to the mouth of the lower tunnel, crossing on the way the mouth of the upper tunnel. An oval valve seat, 4 ft. long, 2 ft. wide, of cast iron, imbedded in concrete, forms the portal of each tunnel. The lower one is set at an angle of 45°, the upper one on the angle of the slope, about 18° from the horizon. The valves to cover these openings are hung on a low four-wheeled car suspended by hangers, which, standing on the slope, swing the valve off the center sufficiently to clear the track. On reaching the seat, a stop arrests the valve, while the car moves on until the hangers are perpendicular to the frame of the car, which allows the valve to drop to its position. The car is raised and lowered by a cable that rests in loose pulleys with deep flanges, placed at intervals between the tracks. When the valve is to be raised, it can be brought to the surface and a screen lowered in its place. The device is simple and operates easily. It was preferred to a masonry tower standing out in the lake where it would

be exposed to severe ice thrust, and might at times be very difficult of access.

A second outlet to the lake was provided on the north side, by means of a tunnel 500 ft. long, at the same level as the upper tunnel of the main outlet, and a gate shaft of the same diameter as the one described, and similarly arranged, was erected. This is designed for future connection with a direct pipe line projected to the Ashland Avenue reservoirs. Both shafts are provided with inlets at the level of the present ground surface, which will draw water 20 ft. from the top when the dikes are completed and the lake is full.

The Third Conduit.—The water-shed of Marston Lake is not over three square miles in area, and the lake has no means of being filled except by the construction of feeders to it. To supply the reservoir, a conduit of 60 000 000 galls. daily capacity is projected to be built from the Platte River. This conduit will be 11½ miles long, of which nearly 3 miles will be in tunnels, 10 in number, through earth or soft sandstone. Wood pipe will be used for the remainder. Pending the completion of this conduit, a temporary arrangement for filling the lake has been effected for the use of an irrigation canal, leading from the cañon of Bear Creek, an important tributary of the Platte. A flume, 1⅓ miles in length, with a capacity of 80 cu. ft. per second, has been built from the canal to the lake.

The waters of the Platte River are so entirely appropriated for irrigation during seven months of the year, that, in order to maintain a constant supply flowing into Marston Lake, a mountain reservoir on the head waters of the stream has been determined upon as a necessary part of the water works, to store the surplus flood water of early summer and the unused flow of winter. A number of sites of large capacity are available, and active work upon a masonry dam of generous proportions will be one of the next items of construction to be undertaken.

When completed, as it has been outlined in these pages, the system will be one of the most perfect in the United States, for amplitude of supply for a much larger population than the city now has, and for purity, clearness and general excellence of quality. The two conduits already built have a combined capacity of 24 000 000 galls. daily from the river, or 30 000 000 galls. if part of the supply be drawn from the higher level of Marston Lake Reservoir, while the extension of the

LAND'S

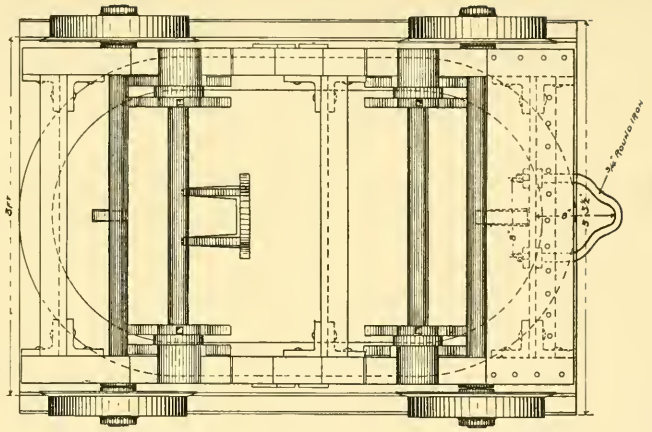
Railroad Gate.

RECONSTRUCTED BY

Chas. P. Allen,

Chief Engr. Citizens Water Co.

Scale 1½ = 1 ft. DENVER, COLO.



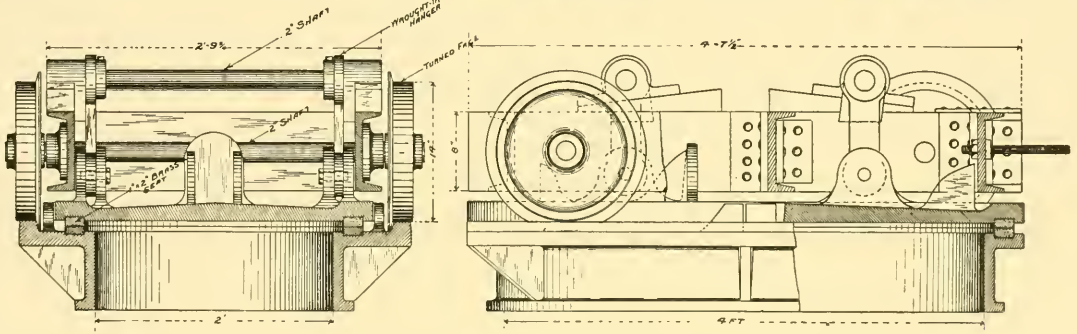


PLATE XIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 691.
SCHUYLER ON WATER WORKS OF DENVER.

third conduit from Marston Lake to the Ashland Avenue reservoirs, will have a capacity of 20 000 000 galls. daily. As the latter is but nine miles long and can be built in a few months whenever needed, it is evident that no danger of water famine need ever threaten the city, after the main feeder to the lake is completed.

The present consumption of water in Denver is enormous, reaching fully 200 galls. daily per capita in the summer and fall months. This is, doubtless, largely due to the reckless waste stimulated by "free water," which the exigencies of sharp competition between the two companies have enabled the citizens to enjoy for two years. The normal consumption, under ordinary prudential use, even in a climate as arid as that of Colorado, will probably not exceed 125 galls. per capita. The present population of Denver is about 150 000, and the summer consumption is estimated at 30 000 000 galls. per day. Even if this rate be continued and increased, the works of The Citizens' Water Company, completed and under construction, will amply provide for the needs of the growing city. The entire plant, when completed to the extent now contemplated, will represent an outlay of about $3 000 000. The company has no indebtedness, no interest charges to meet and small operating expenses. Its rival has an outstanding debt of $4 200 000, and is in the hands of a receiver. Under these conditions, the consolidation of the two companies may be shortly anticipated, and the novelty of "free water with a chromo" brought to an end.

DISCUSSION.

R. C. Gemmell, Assoc. M. Am. Soc. C. E. (by letter).—It may not be out of order to discuss a few of the practical details of construction of wood stave pipe, such as described by Mr. Schuyler. In view of the rapidly increasing use of this pipe and the widely varying opinions in regard to the best methods of construction, the subject certainly is an important one. On the Pacific slope, at least, where such excellent lumber is so plentiful and cheap, stave pipe will, undoubtedly, to a great extent, be used instead of iron pipe for water-works conduits, and instead of box flumes on irrigation systems and water-power plants. It is not because I feel that I am particularly competent to discuss the subject that I bring it up; on the contrary, it is simply with a desire

to provoke a discussion that will bring out all the knowledge and experience of other members of the Society bearing in any way upon the points mentioned below.

In regard to the material used in making the staves: First, should it be wholly free from knots ? and second, should it be thoroughly seasoned ? It is claimed by some makers that small knots, where they extend through and not across the stick, do not occur oftener than one knot in every 2 ft. in length of stave, and none within 1 ft. of the end of the stave, are not so objectionable as to justify the expense of requiring perfectly clear stuff. If the pipe is to be subjected to considerable pressure, however, it would seem to be good economy to use clear lumber. As to seasoning, Mr. Schuyler states that "lumber must be thoroughly dry and seasoned before milling, and after milling the sooner it is built into the pipe the better." For redwood this may be all right. Redwood, in addition to being very soft, possesses the property of shrinking (or swelling, as the case may be) very slightly transversely and very considerably longitudinally. By using it well seasoned, then, there may be no particular danger of setting up any great strain in the bands after the water is turned on, and, perhaps, the longitudinal expansion would only serve to make the butt joints all the tighter. For some work in which red fir is to be used, however, the requirement is that the lumber shall be cut, milled and built into the pipe as quickly as possible. Experience shows that the shrinkage of red fir from a green state to even a partial state of seasoning is fully 4%; hence, it is argued that the only safety in using it without special appliances for taking up the swelling is in putting it in green. Exception may be taken to this by some, however, by reason of the fact that green red fir seasons and shrinks to a certain extent even under water, thus, perhaps, causing loose joints and leakage in the pipe; and also, because of the danger of the sap in the wood, causing decay. With the pipe under considerable pressure, perhaps neither of these objections would hold, as it might be said that the water would force out the sap and penetrate the pores of the wood so thoroughly as to effectually prevent any shrinkage.

For preserving the wood an outside coating of hot asphaltum for the staves is advocated by some, while others think it unnecessary and therefore a needless expense. Those in favor of it claim that it makes an impervious coating, which preserves the surface of the staves and prevents the evaporation of the water that penetrates the pores of the wood from the inside. Those against it claim that the water itself, by completely saturating the wood, is a sufficient preservative, and that the floating of the staves in a bath of asphaltum is sure to leave some asphaltum on the radial edges and the ends, making it difficult to get tight joints; while if the coating is done with a brush, it is very hard to make the asphaltum stick to the wood. This difference gives rise to

another difference of opinion, namely, as to what constitutes the proper thickness for the staves. It is admitted by both sides that the danger of decay is confined to the outside of the pipe, where, under light pressures, the question of saturation may not be without uncertainty for a thick stave. Starting with a safe deflection for a thick new stave, if decay does take place, there may be eventually an excessive deflection, to say nothing of the leakage resulting from the loosening of the bands. On the other hand, the velocity of the water in most cases is high, and if the staves are thin when first put in and the water not clear, there may be so much wear on the inside as to at last cause trouble. My own judgment would be to thoroughly coat the staves on the outside with hot asphaltum, put on with a brush; and to make the staves thick enough to absolutely prevent any appreciable deflection, when using bands of the most economical sizes, spaced for a factor of safety of four.

In making the butt joints sometimes wooden splines are used, instead of the metallic tongues described by Mr. Schuyler. Each method has, of course, its advocates. Those in favor of the metallic tongues claim that in using the splines it is hard to made a good fit, while those in favor of the splines claim that the acid in fir wood will corrode the metal tongues, causing rust and decay. Probably either method makes a good joint. Some builders even go so far as to claim that neither the tongues nor splines are necessary in order to made tight joints. By some it is required that the saddles shall not be closer than a certain distance to butt joints, while others think it is of no importance how the saddles are placed.

The statement that "it is desirable to tighten the bands until they are imbedded in the wood" probably refers to cases in which the material used for making the staves is redwood or some other very soft wood. In order that the seams may be tight, it is necessary that the staves should press against each other with a pressure per square inch at least somewhat in excess of the pressure of the water. This point is an important one, and the cinching can probably be properly done only by skilled and experienced workmen. Perhaps a good method would be not to cinch the bands up very tight at first, turn the water on and allow the wood to swell, and then adjust the tension on the bands properly. That the joints may be tight, and still that no appreciable strain may be put on the bands on account of the swelling of the staves, "is a consummation devotedly to be wished," and worthy of the most careful consideration. In the use of comparatively hard unyielding timber in a seasoned condition, and subject to a lateral expansion of several per cent. while in the process of saturating, it would appear that, with the bands tightly drawn at the time of construction, such swelling might strain the bands beyond the limit of their elasticity. That such danger is recognized by some

engineers engaged in the building of pipe of this character is evident from one having sought a solution of the difficulty by inserting a number of corrugations in the band. It would seem that this effort to obviate the difficulty would require very careful adjustment of the corrugations, and some practical experimenting, to determine just what character of corrugation would remain wholly unyielding under the working strain, and yet yield sufficiently to take up the swelling well before the limit of elasticity of the band was reached. Again, with strains existing in the bands just sufficient to cause a yielding in the corrugations, would not any additional and unusual increase in the water pressure produce an increased permanent yielding, leaving the bands more or less loose after the unusual pressure had subsided, and thereby causing leakage in the pipe ?

Mr. Schuyler is certainly very conservative in estimating the life of the pipe at "from 15 to 20 years," as with the bands and saddles properly coated it would not seem unreasonable, in the light of past experience, to expect the pipe to last fully twice as long. Some builders will also be likely to take exception to the statement that "the bands are the only perishable portion." Mr. B. H. Hull is on record as having stated that the pipe "wears out on the inside," the rapidity of such wear depending to a great extent upon the condition of the water, whether clear or laden with silt or grit.

A question frequently asked is: "What is the maximum pressure under which this pipe can be used?" Generally the answer is that the bands can be proportioned and spaced to stand almost any pressure, and that all depends upon the hardness and strength of the lumber. The prevailing idea among engineers seems to be that, for a safe limit, a pressure of a little less than 100 lbs. per square inch should not be exceeded. The probability is that this pipe can be safely used to all pressures under which it can successfully compete with iron pipe in the matter of first cost.

The above points regarding seasoning, coating, jointing and cinching, as well as those of the probable life of the pipe and maximum pressure to which it may be subjected, have been mentioned because there seems to be such a variety of opinions in regard to them. At least two members of the Society are interested in the manufacture of wood stave pipe. It is to be hoped that they, and any others who may have had occasion to give the matter study, or may have had actual experience in the construction of the pipe, will give us the benefit of their knowledge in relation to mooted points.

A. Fteley, M. Am. Soc. C. E.—In my personal experience I have had more to do with iron pipes, and the question of cast-iron *vs.* wrought-iron or steel pipes has been more conspicuous in this part of the country. My knowledge of wooden pipes for water supplies of cities and towns is almost entirely limited to those of small diameter, which

have been, and, I understand, are still being, made of perforated logs, sometimes strengthened with thin spiral iron wrappings. The long life of these pipes, without any additional supports, is attested by the samples in good preservation, which are, even now, dug out from the streets of New York, and which were a part of its early supply system. It is well known, also, that in our neighborhood penstocks have been made successfully in many mills of wooden staves hooped with iron. It would be consequently difficult for me to speak knowingly of the merits of the pipes just described, but I do not doubt that in localities where proper timber can be found in abundance, and where transportation is expensive, they can be profitably substituted for iron pipes.

Mr. Schuyler's paper shows the prominent fact that an unusual number of successive water-works schemes have been built in Denver. That the first system should have been established on a small scale at a time when nobody could predict the large development of the city, is easily understood; but as soon as the rate of increase of the population, and the establishment of local industries, indicated the almost certain growth of the community, it would have been much wiser to provide for the future, under good engineering advice, such system as would have been susceptible of progressive development, thus securing the best result at the least cost.

This means of foresight has been common to a large number of communities, and shows once more the advantage of proceeding from the beginning in a more rational way.

This paper recalls also the fact that many people are in utter ignorance of the capabilities of water sources. When water is to be collected from the surface of the ground, the capabilities of the watershed to yield water are generally well understood; but when it is a question of underground sources, the apparent mystery attached to the matter sometimes induces the wildest speculations. From a comparison with such works as are now in existence, and from the local circumstances of each case, a reasonable estimate of the capacity of the source in hand can reasonably be made. In places where the amount of water required is small, and where the collecting and storing grounds are extensive, the loss of storage in the underground reservoir from year to year may appear so small as not to be sufficiently noted, and the conclusion may be hastily drawn that the source of supply is practically inexhaustible; but we know by experience that after a time the loss of storage is likely to be felt disastrously, especially when (as is almost invariably the case) the draft from the underground sources increases gradually. To illustrate the extravagant expectations sometimes entertained by communities as to the results to be obtained at a comparatively small cost from underground sources, I may give an instance where a supply of 100 000 000 galls. per day was confidently ex-

pected by a considerable portion of an important population, from underground sources, while an impartial survey indicated a possible maximum of 5 000 000 galls.

James Owen, M. Am. Soc. C. E.—I do not know, Mr. Chairman, that I have had any experience in wooden pipes, but I have had some little groping underground for underground water. There is one curious point about underground supplies that struck me. In the district where I have my practice the water is almost entirely from the red sandstone, and the uniformity of its delivery is, to a certain extent, remarkable. An ordinary driven bored well, 4 ins. in diameter, carried down to 70 or 100 ft., will deliver 30 000 galls. a day. On visiting, some years ago, the Brooklyn Water Works, and asking the engineer, Mr. Van Buren, what he estimated as the delivery of each of his driven wells in the sand of Long Island, he gave me the same figures, that he estimated the delivery of each of those wells nearly at 30 000 galls. a day; it is a curious coincidence.

There is one point that I wish to allude to in this question of underground supply that has not been much touched upon; that is, in reference to the normal delivery of a well from which the water is continuously taken. At the first instance of pumping, the delivery, of course, is much greater than may be expected in the future. As time goes on and the pumping is continued, there is a probability that the permanent delivery may be more than the apparent delivery for the first few years, from the fact that you are creating a current through the soil for the rain water, and although after three or four years the wells may show a remarkable lessening of capacity, yet that may not be the true capacity. The instance I know best was the well put down in Brooklyn Park in 1868, where the first delivery amounted to about 670 000 galls. a day; by pumping for a few years that delivery was reduced to about 400 000 galls. a day, which came to be its normal delivery. I have not seen lately what the statistics show of the delivery, but I think it is fair to presume that it will never be less than that.

In my experience I noted one peculiar thing: I have a plant in which I am using eight wells in the sandstone, and in dry weather the shortness of the delivery becomes apparent with the lack of rainfall, but after a rainfall there was no perceptible increase of flow in the wells until the third day, and on the third day after the storm we always began to appreciate the increase of flow due to the preceding rainfall. It gave me an idea of the velocity with which the surface water would be carried down through the overlying soil, about 20 ft. in depth, until it arrived at the rock and flowed out there.

I would state another instance of underground delivery that occurred in the city of Newark some 20 years ago. When the original plan of the water works was designed, it was proposed to construct

two large filtering beds alongside the river ; they were constructed for that purpose, and they are there to-day. The idea was not to take the river water, but to take the underground water adjacent to the river and pump that for the delivery to the city. The works were built and pumps started, to deliver that water alone. The capacity grew very inadequate for the requirements of the city, and the gates were quietly opened from these chambers, and the river water and the spring water were used together. Lapse of time led people to forget, partly, the ideas that were in vogue then, and the existence of these large beds were partly forgotten, and about 10 or 12 years after that there came to be a craze for the driven well system, and they worked up a sentiment in favor of the driven wells, and the driven-well people put a plant alongside the original filter beds, with the idea that they would be able to deliver 20 000 000 or 30 000 000 galls. a day to the city. Pipes were put down and pumps connected with them, and the amount taken from the driven wells was exactly the amount furnished by the intercepting wells.

I would state that the area tributary to the wells was about one and one-half miles long, with an average of three-fourths wide; to the south it was entirely intercepted by a high ridge, which prevented the water from flowing that way; on the west was a trap-rock ridge, and on the east and north by another water-shed entirely.

L. L. Buck, M. Am. Soc. C. E.—What was the depth of these wells ?

Mr. Owen.—The average was about 100 ft.; we had one large well which ran down 50 ft., and three diamond drill borings went 50 ft. farther. One went down 107 ft.; that was in red sandstone.

That was entirely through gravel. The sandstone has the lines of cleavage and lines of stratification. The water will percolate much more freely than through sand.

Edward P. North, M. Am. Soc. C. E.—Mr. President, I had in Mexico a little experience with driven wells which was new to me, and I have not seen anybody that has had a like experience. On a very flat shore, which was mostly clay, there were some sand dunes, and in the center of these dunes was a depression which always contained fresh water, the surface of which was only 1 ft. or so above the level of the salt water. The sand dunes came down to the seashore, and I concluded that they had acted as a condenser of the dew and the moisture contained in the air, and the fresh water was lying on top of the salt water because it was lighter. I drove wells, and found I could get fresh water about 6 ins. above high-tide level; but after we had pumped for the use of thirty men for about two days, the well would get brackish; it would be necessary to move the well some 20 or 30 ft., when we would get fairly fresh water for some two days more. The well was moved around in this way, and that was the main source of supply for some

three or four months, saving us bringing water some two miles. There was one other place on the shore where fresh water was obtained from a sand dune, that had on one side of it salt water and on the other side a salt marsh; whether the fresh water would be exhausted by a little pumping I do not know; it probably would have been.

E. E. Olcott, M. Am. Soc. S. E.—Seeing that Mr. North takes up Mexican experiences, I am rather at home. I had an experience at one time in striking water in a dry cañon, to the wonderment of the Mexicans, where quite a large supply was needed. It was a mooted point for some time as to where the supply had best come from, and by studying the direction of the bed-rock at the side of the valley, which was entirely dry for the greater portion of the year, I judged that we could strike water in the sands beneath. By sinking a well 35 ft. deep and running drifts out across the channel, we did strike a very large stream of water which was successfully pumped up through $1\frac{1}{2}$ miles of pipe to an elevation of 575 ft., the pumps used being duplex compound Worthingtons.

In regard to the wooden stave pipe, I consider that the promise for its successful introduction is very good in the West, especially in California; it is used already in connection with irrigation enterprises. I think that if it had been introduced during the time of hydraulic mining in California it would have been largely employed, as it offers advantages over iron pipe for bringing water to the hydraulic claims. The abundance of splendid timber there would have made it very cheap. An advantage this wooden pipe offers in some work is the facility with which it can be tapered and reduced from one size to another and connections made.

I was surprised to see the statement that the percolation through sand was not much more rapid than from $\frac{1}{2}$ mile to 2 miles per annum.

I was interested in another point in the paper, viz., the scraping out the sediment from the reservoir. I saw an interesting case in Colombia, S. A., where the healthfulness of a locality was destroyed by flooding a large basin without first clearing away the vegetable matter. That is something that engineers are not careful enough about; even the vegetable mold should be entirely scraped up and removed as thoroughly as possible.

L. L. Tribus, Assoc. M. Am. Soc. C. E.—I can, perhaps, throw a little additional light on two points in this subject. In the southern portion of New Jersey there are a good many driven wells of about 50 ft. in depth. The rainfall is rarely felt in them under about 30 hours, while it takes a very heavy storm to show inside of 24 hours.

This indicates a vertical filtration of about $3\frac{1}{4}$ miles per annum.

As to the other point, the durability of wooden pipe; a few years ago I had occasion to take out wooden water pipe that had lain in the ground for nearly 40 years, part of that time used in the water supply,

part of the time dry, the ground itself, sometimes wet, sometimes dry, and yet the logs were in perfect preservation. They seemed to be yellow pine.

Mr. North.—What soil was that ?

Mr. Tribus.—In sandy soil, near Mobile, Ala.

Mr. Fteley.—Mr. Chairman, it seems to me that Mr. Gleim could give us some interesting comments on the subject, as he is connected with the city of Hamburg, and we have all heard of the works for pipe-laying in that city. We would be very glad to hear from him.

C. O. Gleim, Cor. M. Am. Soc. C. E.—I am not prepared to speak on the subject of, and, not having anything to do myself directly with, those works, I am only half familiar with them. I do not see exactly that it has a great deal to do with the subject under discussion this evening, except that it is water works. The city of Hamburg has drawn its supply from the River Elbe ever since the time of the great fire, about 51 years ago. The water works had been planned at the time the city was burned down, and the works were commenced immediately. Originally they were planned with a view to guard against another case of fire ; it was not at that time meant as a source of supply for house use, but it was afterwards also used for that purpose. The water from the river was let into large subsiding reservoirs, and from them drawn into the city. But from a very early time it had been contemplated to introduce filter basins so as to improve the quality of the water for potable purposes. The introduction of the filtration, however, has been delayed through various causes, for a long time. There was at one time a scheme brought up by one of the members of the Common Council, a doctor of medicine, to introduce what he called a peripheric system of filtration ; that is to say, not to have one central system, but to have local systems of filtration. It was because of the effort in that direction that the scheme for central filtration was delayed. Another scheme was to draw water from lakes in the Duchy of Holstein in the neighborhood, which, however, was found to be inadequate ; either it did not furnish a sufficient amount of water, or the water was of inferior quality; and if it had been satisfactory, there were difficulties in obtaining the water, owing to the political boundaries. So, after all these delays, the authorities at last voted the money to build the great central filtration scheme, which, by the scale of expenses that we are used to in Germany, was quite an expensive one. The amount of the first appropriation was 7 000 000 marks, and that has been entirely exhausted, requiring some additional appropriations.

The old intake for the water works was about four miles above the inlet of the effluent of our sewerage, which empties into the River Elbe below the city. As you all know, the river is subject to ebb and flood, and the tide current takes the polluted water from this effluent

of the sewage up to the river, but only to a point some distance below where the intake used to be. In the present filtration scheme that intake has been taken still further up the river, about 1½ miles I should say. We have now an excellent supply. The works were two-thirds finished when the cholera came last year; it came one year too early, or it would have found us better prepared. The water was very poor before this filtration scheme was carried out. It would have been of better quality if we had not thought of filtration, because the city would have had to enlarge the settling reservoirs; but with this scheme in view, we have never spent any money in this way. Since last June we have the filtered water, and from the reports that I have seen in the newspapers, the few cases of cholera that are now there are attributed to remnants of the poor old water in the pipe system and probably to the incrustations of the old pipes, which will require a certain time to get rid of; but I believe that with the present system the city will not be subjected to any such large epidemic as we have had last year.

J. D. Fouquet, M. Am. Soc. C. E.—I might give some information as to the durability of wooden flumes, made after the barrel type, as I have made some investigation on the subject. At Wicopee on Fishkill Creek are located the New York Rubber Company's mills, and just above the works is a dam furnishing water-power. Just below the rubber works is located a grinding mill, erected many years ago by John and William Rothery, manufacturers of files. Power is brought to the grinding mill through 300 ft. of a wooden cylindrical trunk, 2½ ft. in diameter; staves were made of the best clear white pine, 5 ins. wide, and about 1¾ ins. in thickness. Iron hoops are arranged about 2 ft. apart. The trunk is supported on wooden trestle bents, which have been repaired from time to time. The trunk which was built in 1850, Mr. Rothery assures me, is in good condition at the present time, and is good for ten years more of service.

Another shorter trunk at Carrol's straw hat works, about 1½ miles above this, is partly under and partly above ground; was built about twenty years ago, and is now in excellent condition.

Hoops are arranged to cover the joints of staves, which are spaced to suit the length of the stave plank.

R. L. Harris, M. Am. Soc. C. E.—As the durability of wooden water pipes seems to be an interesting point this evening, I would say that the city of Portsmouth, N. H., has good examples.

The original water pipes were bored pine "pump logs," joined by taper point and socket.

Some of these pine logs of 5-in. bore, laid so early as 1797, were removed in 1870. These were found to be sound, but were worn on the inside to diameters of about 8 ins., as was well shown by the smoothly polished knot wood projecting to the inside, the softer wood having

been worn away by the water. The location of these logs was in a main one mile out of town, through wet meadow-land.

In this connection the following may be of interest: Mr. Oliver Ayres was Superintendent of Portsmouth Aqueduct for 50 years previous to 1884. Mr. Joseph Akerman was superintendent for 30 years before Mr. Ayres. Mr. Akerman told Mr. Ayres that he had never seen the service pipe to the colonial mansion of Governor Langdon. This service pipe, a pine pump log, was uncovered after 1875, by reason of a stoppage; it was found to be sound, a root seeking water had grown through a joint and had blocked the bore; this pipe was in wet clay about 5½ ft. beneath the surface. Here, then, are two examples of sound pine log pipes of over 70 years of age, both a main and a service pipe.

In gravely hills the pipes were replaced when 30 to 40 years old.

As to the matter of removing vegetation, we are all familiar with the fact that it is necessary to remove this, but a special case has come under my observation this summer where a city of 40 000 people built water works two years ago and are now paying for drinking water 5 cents a gallon, the reason being that the vegetable matter was not removed from the reservoir when it was built.

J. D. Schuyler, M. Am. Soc. C. E. (by letter).—As Mr. North's successor in Mexico I had occasion to make a study of the sand-dune water supply to which he refers, with the view of utilizing it to supply locomotives at the western terminus of the Sinaloa and Durango Railroad, and came to the conclusion that the source of supply was entirely independent of the rainfall and was probably due to condensation of moisture from the air which penetrated the sand far enough to reach a much lower temperature. The supply was constant and reliable, but limited in volume, and was apparently in direct ratio to the size of the sand pile. A similar phenomenon is observable in San Francisco, where the sand dunes are piled up to a much greater height and yield larger volumes of water. The first water works of San Francisco derived their supply from one of these little sand-dune lakes, near the Marine Hospital, and so near the sea level that the flume carrying the water to the city skirts the shore all the way just above tide water. This is still in service. Laguna Honda, from which several million gallons daily are pumped to the city, is another sand-dune lake near the ocean level and separated from it by a narrow sand spit. Neither of these lakes has a water-shed large enough to account for the supply which is derived from them as possibly coming from that source, and I have no doubt that they are perpetually replenished by condensation of moisture in the atmosphere.

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692.

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TRAIN LOADINGS FOR RAILROAD BRIDGES.

By Theodore Cooper, M. Am. Soc. C. E.

Read January 3d, 1894.

WITH DISCUSSION.

The great advance made within recent years in the matter of metallic bridge construction, and the gradual convergence of opinions towards uniformity in our practice, has led many to hope for and believe in a final solution of all the various minor points connected therewith. The writer never has accepted any such idea, and does not believe that we have reached the end of our education or development in any direction. The tendency to seek for a "final standard" in any engineering construction, regardless of changing conditions, is a harmful one. Take from the engineer the right of professional judgment, and how does he differ from an automaton? While the writer does not believe that all the disputed points in engineering practice can ever be brought to positive uniformity, we can, by fair and thoughtful discussion, approach it more and more. To do this in the most effectual

manner, it will be better to limit the discussion so far as is practicable to one subject-matter at a time.

The one subject upon which there is more general interest than any other connected with railroad bridges is the character of the live loading to be specified, and upon which there is a far greater probability of harmony than many others.

Which mode of obtaining the strains, due to the finally selected loading, is adopted, is a matter of minor importance, provided the results are close approximations to the true strains produced by the standard loading; for we must, for fair comparisons between bidders, have some standard of final reference in case of difference, and there is none that can be generally selected which would be simpler or more positive than an actual train load.

Throwing aside, therefore, all references to relative merits of actual strains, diagrams, tables or other methods of rapidly approximating to the truth, let us take up and simplify the train-loads by reducing them to their simplest forms and least number to cover the necessities of various classes of railroads.

The miscellaneous collection of train diagrams now in use has grown tentatively and naturally till it has reached its present absurd condition. The first engines selected were properly the heaviest actually existing engines, with the useless refinements of all their fractional dimensions and weights. In no other way could the managers of our roads have been made to pass from the uniform loads then in general use to a higher loading for their bridges. No theoretical or badly proportioned engine would have been accepted. Such would have been considered as evidence of the ignorance or impracticability of the engineer, not only as to the proper proportions of a real engine, but as to the desirability of a more suitable train load by which to proportion their bridges.

That it was the most important advance towards better bridges will not be questioned by any one having knowledge of the past history of railroad bridge building. As this step forward was necessary upon all our various railroad systems, it was natural that each engineer selected the engine and train best suited to meet the circumstances of his own case. The next step was the inventing of typical engines to represent the expected developments in the future traffic of the various roads. The result is naturally one of confusion; but, nevertheless, it

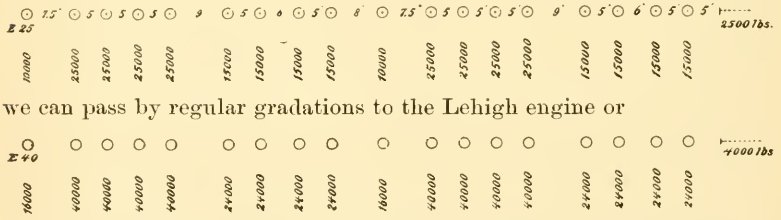
was a necessary evil for the time being. The ultimate purpose has now been accomplished and we can now readily reduce order out of chaos by the selection of a few typical train loads sufficient to cover all practical cases.

We can all agree upon the uselessness of refinements in dimensions or weights.

It would also seem possible to agree upon two types of engines to represent the loadings produced by passenger and freight trains with varying wheel loads to suit each special case.

We can also accept, without question, the freight train loads as the only car loadings necessary to consider, for the heaviest passenger car (114 000 lbs.), crowded with people, will not give a load per lineal foot of track over 2 000 lbs., nor a wheel load exceeding 12 000 lbs.

If the above general statements can be accepted, it would appear to the writer possible to reach a harmonious conclusion as to the standard train loads suitable for different railroads, and that such train loads can be arranged so as to be changeable by regular gradations to any extent, thus meeting all present conditions and providing for any future ones, without introduction of any complications in methods of computation other than uniform increase by percentages. Then each computor, who has his special method of determining the strains due to any train loading, whether by tables, diagrams or otherwise, can readily change to any new condition by increasing all the strains by a fixed proportion. For example, if we assumed the following diagram as representing the standard freight engine and train for minimum loading:



we can pass by regular gradations to the Lehigh engine or

to cover different conditions, without departing widely from the proportions of actual engines.

As the loadings throughout the whole train are all fixed proportionally to the governing load on the drivers, the strains in each case will also be directly proportional to this load; therefore, all strains due to

any engine and loading of the series can be obtained proportionally from the strains due to Engine 25.

The foregoing diagram for the passenger engine can be increased in like manner. If, then, we label our minimum train-loading E. 25, we can adopt a nomenclature readily understood by any one; as, for example, E. 30 would be a train load 20% heavier than E. 25, E. 35 40% heavier, etc. The live loads in the two cases being in like manner 20 and 40% heavier than those for E. 25.*

The previous diagrams are only offered at present as illustrative of the method proposed by the writer. If that be accepted as a desirable one, the exact values to be selected for the several weights forming the minimum train loading can be more readily reached by mutual discussion.

As a preparatory step towards this discussion, the writer has collated certain data to be attached to this article as bearing upon the possible future train loads.

The great increase in the weight of engines and car loads during the past few years has impressed many with the belief that the increase for the future may be at a similar rate. This is hardly possible, as we shall endeavor to show. There are several factors that tend to limit the increase to a possible determinate maximum, as far as the load per lineal foot of track is concerned, and this is the principal point of train loadings. These factors are the limitations of the cross-section of the car-body, the capacity of the axles or journals, and the methods of loading the cars.

In like manner, the advance in weight of the engines is controlled by the maximum load on one driver and numerous constructive features of the general machine. As the engine is practically limited to one line of railroad and often to a single division of such railroad, the restrictions are not so general as for cars which are liable to pass over wide ranges. The peculiar features of special roads, such as the roadbed, rails, curves and so forth, may allow the use of engines totally inadmissible upon other systems. But that there is an economical limit to the size of our engines is certain, for, not only can we see

*The writer has had occasion recently to use this percentage method, being asked by the general manager of a railroad to specify an engine and train-loading considerably heavier than his class, heavy A, but not as heavy as the Lehigh; the live load was changed to one, 20% greater than heavy A, or the strains produced by heavy A were increased 20 per cent.

a limitation to the engine wheel loads in spite of improved roadbeds and heavier rails, but also a final compromise in the existing struggle between the tractive power of the engine and the draft mechanism of the cars. The destruction of freight trains on most of our mountain divisions through the breaking away of the train, from failure of the draft mechanism to equal the hauling capacity of the engines, must wipe out an important part, if not all, of the gain hoped for by the use of much heavier engines.

As for the cars, these, by the necessities of ready exchange from road to road without transfer of freight, must be adapted to the general limitations of the general railroad system of the continent; hence, there are limitations to the width and height of such cars. The width of cars cannot pass certain accepted limits without affecting the clearances at existing bridges, tunnels, freight houses, station platforms, etc., of all roads; and for double-track roads it would require increased width between tracks, involving wider cuts, embankments, tunnels, bridges and other changes. Increased height, also, would involve changes in tunnels, bridges, etc.

We can therefore assume that radical changes in the width and height of cars are not very probable from the present outlook of clearances. Moreover a careful consideration of the fact that we are not now using the full cross-section of our car-bodies for more than a small percentage of the freight handled by the railroads, on account of the great loads that would be imposed upon the journals and the greatly increased strength of frame needed, is an additional reason for believing that wider and higher cars are not probable.

The cross-section of the average box car in general use does not exceed 55 to 60 sq. ft.

The largest car-body known to the writer is the 40-ft. furniture car of the Chicago, Burlington and Quincy Railroad (see *Railroad Gazette*, January 27th, 1893). This has an inside cross-section of 74 sq. ft.; but only 3 ft. and 4 ins. in depth is allowed to be used for wheat and corn (capacity limited to 50 000 lbs.), its full inside height being 8 ft. 9 ins.

If this car were built strong enough to be filled with wheat or corn, retaining the same outside dimensions, the useful cross-section would not be more than 70 sq. ft.

Such a car fully loaded with corn or wheat would have a capacity of 130 000 lbs., far in excess of any car now in use; it would, however,

only impose a load of 4 000 lbs. per foot of track, allowing 1 000 lbs. per foot for the weight of the car itself.

In Table No. 1 the writer has collected the general dimensions of the heavier cars in actual use to-day upon American railroads, with the loads they impose upon the track per lineal foot and upon each axle. It will be seen that, with the exception of the gun cars, no cars are intended to impose a load as high as 4 000 lbs. per foot of track, or more than about 24 000 lbs. upon a single axle.

Table No. 2 gives a summary of the freight handled by a number of the most important roads (many roads do not give any classification of the freight handled by them), which we may accept as an approximate guide in consideration of this subject.

In Table No. 3 is given the weight per cubic foot of the principal articles other than ores and metals, measured on the space occupied by such articles as usually stored.

In Table No. 4 are gathered the weights per square foot of floor space of a number of representative classes of machines. A careful study of these tables appears to the writer to justify the following line of reasoning:

The average box car, which may be taken as best representing the car of greatest storage capacity, has about 33 ft. of effective storage length, and occupies about 37 ft. of track. With a possible maximum cross-section of 70 sq. ft., the storage capacity of such a car would be 2 310 cu. ft.

If this space could be fully utilized for the heavier coals (56 lbs.), the carrying capacity of the car would be 130 000 lbs. and the load per running foot of track (allowing 1 000 lbs. per foot for weight of the car) would be 4 500 lbs.

Loaded with corn or wheat, the capacity would be 111 000 lbs. and the load on the track would be 4 000 lbs. per foot.

It would be impossible to exceed these loads by a large part of the "machinery," "castings," "structural iron-work" and "stone," on account of the difficulties of storing them on the cars in a compact shape.

If, therefore, we consider the difficulties of loading and fully storing such cars, it would appear that 4 000 lbs. per foot of track will cover at least 90% of the freight handled by the general railroad system of the country.

Such a car as we have assumed would weigh, loaded, 148 000 lbs. This load would give 37 000 lbs. on each axle for four axles, or 24 666 lbs. for six axles, which would require two 6-wheeled trucks under each car, or three 4-wheeled trucks. While this last method has been employed for certain ore cars, it is hardly likely to become the practice.

In addition to the above facts, it must be borne in mind that cars built for these heavier loads must be made stronger and necessarily heavier; that such heavier cars must frequently be used for the larger body of lighter freight handled, in order to keep up a paying general circulation of rolling stock, and that in such cases we have increased the non-paying tonnage to the detriment of a large body of freight to benefit another class. The natural tendency must therefore be to get cars which will be the best suited for the average rather than the heavier freight.

All things considered, therefore, we think it very improbable that our train loads will ever exceed, as a rule, 4 000 lbs. per foot of track.

No doubt we can also agree that we ought to construct the bridges of even minor or weaker roads up to a standard equal to the diagram E. 25, or can start with this as a minimum loading.

As passenger engines are liable to pass over even minor railroad systems, which may not have a heavy freight traffic, to make a connecting line for some general through passenger route, bridges of all roads should be proportioned for the heavy passenger engine.

While passenger trains are lighter than freight trains per running foot of track, the concentrated loads on the passenger engine frequently induce heavier strains upon the shorter spans or panels than the heavier freight engines.

In addition, therefore, to the freight train and engine, a pair of wheels with a heavy concentrated load should be employed to proportion our bridges.

Passenger engines with 40 000 lbs. on each of two axles are in use, and some designed with 45 000 lbs. on each axle.

Instead of introducing a second diagram to cover this case, it has been the writer's practice to require that two concentrated loads, at a fixed distance apart, shall also be considered.

For this, he requires, for the lighter specifications, that two 40 000-lb. loads, at a distance of 7 ft., must also be provided for, and for the heavier specification, two 50 000-lb. loads.

The writer would, therefore, offer as a basis for discussion the following specification for varying train loads:

1st.	Train load....	E. 25	with two	40 000	lbs.	at 7	ft. distance.
2d.	" " ..	E. 30 / E. 30	"	45 000	"	7	"
3d.	" " ..	E. 35 / E. 35	"	50 000	"	7	"
4th.	" "	E. 40					

TABLE No. 1.

Heavy Car Weights and Loadings.

Kind of Car.	No. of axles.	Length over all.	Weight of car.	Capacity of car.	Total weight loaded.	Load per foot of track.	Load per axle.
P. R. R. hopper gondola	4	29 2	23 200	60 000	83 200	2 853	20 800
E. T. V & Ga. "	4	30 6	25 000	50 000	75 000	2 459	18 750
Ches. & Ohio "	4	30 9	24 000	60 000	84 000	2 732	21 000
Phil. & Read. "	4	35 4	22 800	60 000	82 800	2 343	21 200
Phil. & Read. coal car	4	23 6	18 480	56 000	74 480	3 170	18 620
L. S. & M. S. gondola car	4	35 6	27 150	60 000	87 150	2 455	21 790
L. S. & M. S ore car	6	34 3	36 500	100 000	136 500	3 985	22 750
Duluth & Iron Range ore car	4	24 6	24 400	53 760	78 160	3 190	19 540
L. & N. box car	4	36 8	28 000	60 000	88 000	2 400	22 000
S. P. box car	4	30 9	23 500	50 000	73 500	2 390	18 375
S. P. "	4	36 9	27 000	50 000	77 000	2 095	19 250
N. Y., L. E. & W. box car	4	36 5	31 900	60 000	91 900	2 524	22 975
P. R. R box car	4	37 6	30 000	60 000	90 000	2 400	22 500
C., C. C & St. L. box car	4	38 4	33 500	60 000	93 500	2 460	23 375
Street's stock car	4	40 4	29 800	60 000	89 800	2 226	22 450
Wicke's refrigerator car	4	35 6	42 300	50 000	92 300	2 600	23 075
P. R R. refrigerator car	4	37 6	41 400	60 000	101 400	2 700	25 350
A., T. & St. Fé refrigerator car	4	37 6	41 500	40 000	81 500	2 173	20 375
P. R. R. gun truck, old	8	33	36 100	80 000	116 100	3 518	14 512
" " cable car	8	40 2	51 800	100 000	151 800	3 779	18 975
" " Krupp 120-ton gun	16	90 9	175 000	270 000	445 000	4 903	27 812
" " " 62 ton gun	12	70 9	113 300	140 000	253 300	3 580	21 108
" " single gun car	6	35 3	45 000	100 000	145 000	4 113	24 166
Baldwin's locomotive car	4	33	24 000	70 000	94 000	2 849	21 750

TABLE No. 2.

Classification of Freight Handled by Railroads.

The total freight handled during 12 months, amounting to 152 658 240 tons, upon the following railroads, viz.: New York Central and Hudson River R. R.; Lake Shore R. R.; Pennsylvania R. R.; United R. R.'s of New Jersey; Philadelphia and Erie R. R.; New York, Lake Erie and Western R. R.; Atchison, Topeka and Santa Fé R. R.; St. Louis and

San Francisco R. R.; Union Pacific R. R., and 8 400 miles of railroads in Iowa, is classified as follows :

Coal and coke	66 289 094 tons	44% of total
Grain	13 135 104 "	9% "
Lumber	11 588 910 "	7% "
Merchandise	20 427 453 "	13% "
Farm products	9 491 496 "	6% + "
Animal "	8 193 952 "	6% — "
Petroleum	1 868 190 "	1% "
Ores	5 438 494 "	3.5% "
Pig and bar metal	4 905 372 "	3% "
Mach'y and castings...	3 385 395 "	2% "
Stone, etc	7 934 780 "	5.5% "
Totals	152 658 240	100% "

TABLE No. 3.

Weight of Various Materials, Measured upon the Cubic Space Occupied by Them when Stowed in the Usual Packages.

Materials.	How packed.	Weight.
		lbs.
Coal, bituminous	In bulk	45 to 50
" anthracite	"	49 to 56
Coke	"	26 to 35
Lumber	"	30 to 45
Wheat, peas and beans	"	48
Corn and rye	"	45
Barley	"	38
Oats	"	26
Clover, flax and cotton seed	"	40
Hay	Bales	14
"	Compressed	24
Straw	"	19
Hops	Bales	11
Hemp	"	20
Wool	"	15 to 29
Tow	"	29
Excelsior	"	19
Cotton	"	12
"	Compressed	25 to 40
Leather	Bales	16
Raw hides	"	13
"	Compressed	23
Tobacco, leaf	Hogsheads or boxes	20
" plug	Boxes	56
Flour	Barrels	33
Pork and beef	"	40
Lard and tallow	"	40
Bacon	Bulk	38
Whiskey	Barrels	36
Sugar	"	42 to 45
Salt	"	48
Coffee	Bags	34
Cheese	Bulk	30
Oil cake	"	60
Oils	Barrels	35
Pitch and resin	"	48
Lime	"	50
Cement, American	"	56
" English	"	72
Plaster	"	53
Soda ash	"	62
Glass	Boxes	60
Crockery	Crates	40
Rope	Coils	42
Woolen goods	Bales	13 to 22
Cotton goods	"	16 to 37
Rags	"	15 to 23
Paper, newspaper	Rolls	38
" writing paper	Bundles	64
" wrapping paper	"	97
Nails	Kegs	63 to 74
Spikes	"	58 to 67
Rivets	"	91
Tin	Boxes	278
Railroad iron, 40-lb. rails	In tiers	156
" " 100-lb. "	"	145
Pipe, cast iron, 6 ins. diameter	"	81
" " 12 "	"	62
" " 36 "	"	40
" " 48 "	"	35
" Wrought iron, 1 in. diameter	"	100
" " 3 ins. "	"	53
" " 6 "	"	40
" " 12 "	"	29
Bridge work, top chords	"	50 to 90
" floor beams	"	45 to 75
" pedestal castings	"	50 to 100
" eye-bars	"	325

TABLE No. 4.

Weight of Machinery per Square Foot of Floor Space.

Name of machine.			Weight per square foot floor.
Westinghouse engines,	4	H. P.....	121 lbs.
" "	25	"	178 "
" "	50	"	300 "
" "	100	"	303 "
Lidgerwood's H. engines,	6	"	241 "
" "	20	"	310 "
" "	50	"	361 "
" Up. Tub. boilers	5	"	202 "
" " "	15	"	203 "
" " "	35	"	259 "
" " "	60	"	280 "
" Sta. Loco. "	15	"	153 "
" " "	30	"	166 "
" " "	100	"	179 "
" Hor. Tub. "	20	"	103 "
" " "	100	"	159 "
Clerk gas engines,	5	"	117 "
" "	30	"	120 "
Gardiner's 3 Cyl. engines,	10	"	139 "
" "	60	"	214 "
New Acme Ver. engines,	5	"	156 "
" "	10	"	255 "
" "	20	"	287 "
N. Y. Safety St. Power V. Eng.,	5	"	160 "
" " "	10	"	190 "
" " Ver. boilers,	5	"	216 "
" " "	20	"	222 "
Holly Mfg. Co., heavy press. pumps	No. 1	.....	106 "
" " "	No. 13	.....	176 "
New Pulsometer pump	No. 5	.....	143 "
" "	No. 10	.....	308 "
Terracute Mach. Co. presses		.....	from 100 to 340 "
Bending, Forming and Shearing Mach.		.....	235 "
Nut tappers		.....	84 "
Emery machine		.....	86 "
Punching presses, heavy		.....	340 "

DISCUSSION.

GEORGE H. BLAKELEY, Jun. Am. Soc. C. E.—The discussion of this paper from the standpoint of the manufacturer may not be amiss. The almost endless variety in the existing specifications of live loads for railway bridges is a manifest absurdity of engineering practice that, for a long time, has been a subject of adverse criticism among the bridge builders and the engineers of the bridge building companies, as they have to bear the bulk of the burden which it imposes, and I can assure you it is no small burden.

It has for a long time been apparent to the bridge builders that it would be a comparatively easy task, were the engineers interested so inclined, to frame a standard system of typical loadings for railway bridges that would cover the ground as completely and thoroughly as the existing mass of specifications, and relieve them of a great quantity of entirely unnecessary and laborious calculations. In this matter the builder is at the mercy of the railroad companies. It is not for him to dictate. He must submit to the exactions of the purchaser. It is gratifying then, from our standpoint, to observe this tendency among the bridge engineers to simplify this question of the live loads for railway bridges, and bring order out of the existing chaos of almost infinite variety of specifications differing only by infinitesimal variations.

The paper on "Some Disputed Points in Railway Bridge Designing," by J. A. L. Waddell, M. Am. Soc. C. E., *Transactions*, Vol. XXVI, and which was discussed at great length by the members of this Society, was a long step in the right direction. That paper, however, covered too much ground and laid too great stress on the method of calculating the live load strains, and proposed a radical amendment in existing practice. The consensus of the opinion of those most competent to judge, and with evident correctness, condemns any departure from the employment of anything but typical engine loads in the design of railway bridges, not because of any superior ultimate accuracy that may be gained by their use, but by reason of their furnishing a readily understood standard that can be easily grasped, almost at once, by every railroad manager, being more or less a pictorial representation of actual loads, the comprehension of which is not confined to expert bridge engineers. It would be a serious mistake to abandon the use of typical loads. Having settled upon the retention of the typical engine load, the method of calculating the stresses is a question of minor importance, a secondary consideration, that can be left to the preference of the designer.

The paper presented seems to point the way to a simpler solution of this whole question of the proper live load for railway bridges than

any heretofore proposed. It places the whole question upon a more rational basis than any method yet proposed, and I am glad—heartily glad—that it has the sanction of such an eminent authority as Mr. Cooper. The bridge builders will await, with a great deal of interest, the opinions of the members of this Society as to its ultimate adoption.

In regard to the typical loading to be selected, doubtless we all have our own opinions ; but it does not seem that it will be very difficult to agree on one that will satisfy all, and meet all the requirements demanded ; and one for which every railroad bridge should, at least, be built, or made as much stronger as the requirements of the traffic demand, or the financial condition of the railroad permits. Mr. Cooper's proposed method of increase by percentages has many admirable points of merit which will commend it to every engineer familiar with the subject.

Mr. Cooper presents his proposed typical loading to this Society for general discussion. It is with this understanding that the following suggestions and information, gathered from locomotive builders, in reference to the general distribution of weight in the consolidation type of locomotive is presented for consideration.

The weight on the leading truck seems to be somewhat lighter than usual. The consolidation engine has, in general, about 86% of its total weight on the drivers when two of the driving axles are located under the firebox, and about 89% when the rear driving axle only is under the firebox. That is, the weight on the truck is 14% and 11%, respectively, of the total weight of the engine in the two cases cited. In the proposed loading the truck is only 9% of the total weight of the engine.

As a matter of convenience in calculation, the distance from the leading truck to the first driving axle might be made 8 ft. instead of 7½ ft., and thereby preserve integral wheel spacing throughout.

The driving wheel base of many of the existing consolidation locomotives of the lighter types is about 14 ft., but the mechanical engineer of one of the large locomotive companies assures me that the tendency is to the use of driving wheels 56 ins. in diameter, thus making 15 ft. the minimum rigid wheel base. In this light the use of 5-ft. centers of driving wheel axles seems well chosen as coinciding with the indications of present practice.

I believe that the proposed weight of tender should be increased somewhat to agree with actual conditions. Tenders are constructed at present much heavier, in proportion to the weight of engine, than formerly, and the tendency is to still heavier tenders. The New York, Lake Erie and Western Railroad is at present constructing a number of tenders weighing about 90 000 lbs., loaded, or about 22 500 lbs. on an axle. These tenders will be attached to their regular 55-ton engine, which has 28 000 lbs. on a driving axle. These phenomenally

heavy tenders are designed for use on a portion of the road where the distance between water stations will be longer than usual; still I believe that the weight of tender proposed in this paper should, at least, be increased $6\frac{2}{3}\%$, or so that the tender for engine 25, adopting the nomenclature of Mr. Cooper, would weigh 64 000 lbs., instead of 60 000 lbs. This would be the minimum increase, and I believe that an increase of 10% would not be out of the way. The following table gives the actual weights of several locomotives and the corresponding weights of accompanying tenders:

ACTUAL WEIGHTS OF LOCOMOTIVES.

Railroad.	Type of engine.	Weight on drivers.	Weight on truck.	Total weight of engine.	Weight of tender.
D., L. & W. R. R.	8-wheel Passenger	72 550	29 600	102 150	64 940
" "	" "	76 000	24 000	100 000	80 000
B. & O. R. R.	10-wheel	101 500	26 000	127 500	76 000
D., L. & W. R. R	"	120 000	34 550	154 550	82 700
N. Y., L. E. & W. R. R.	"	111 750	26 350	138 100	79 600
" "	"	96 800	29 000	125 800	81 000
D., L. & W. R. R.	Mogul	97 770	16 200	113 970	75 590
" "	"	105 000	13 000	118 000	69 750
N. Y., L. E. & W. R R.	Consolidation	115 800	15 800	131 600	79 600
" "	"	110 700	19 500	130 200	79 600
C. & O. R. R.	"	110 800	16 500	127 300	72 000
Rogers Loco. Co	"	108 000	12 000	120 000	68 000
W. N. Y. & P. R. R.	"	113 840	13 120	126 960	69 600

Some objection may be urged against the proposal of making the uniform train load a direct function of the weight of the engine, on the ground that there are exceptional cases of small or branch roads, hauling special freight heavier in proportion to the weight of the engine employed than is usual on the large or trunk lines. We must not lose sight of the fact that two consolidation locomotives coupled together, as the specification directs, cover a span of 100 ft., and the uniform train load is not considered in the calculation of the maximum stresses, unless the train load is heavier than the locomotives, a condition of affairs that is seldom or never realized in actual practice. In fact, the effect of the train load on spans of 125 ft., or even on spans of 150 ft., is inconsiderable. In this connection it is well to bear in mind that the great majority of railroad bridges are under 150 ft. span.

The provision for the effect of the heavy concentrations of the eight-wheel passenger locomotive, simply by the specification of two loads spaced a fixed distance apart, is to be commended from the calculator's standpoint, and is an instance of the elegance of the results that can be obtained by the use of the simplest methods. In marked

contrast to this is the practice of some bridge buyers, not content with the specifications of an anomalous loading, demand that their bridge shall be calculated for one or more additional loadings. We should bear in mind that complexity is not always an index of profundity, especially where simplicity serves the same end.

I believe that no railroad bridge, even for small or branch roads, should be constructed for a less load than engine 27 of Mr. Cooper's nomenclature. For some years I have recommended substantially this engine where the matter has been left to my judgment. I wish to urge Mr. Cooper, when he comes to the revision of this paper, to adopt an engine not less than engine 27, as a minimum standard typical loading for railway bridges.

This leads me to a subject that has not received the consideration that it should from the managers of the railways, and possibly many railroad engineers as well. It seems to be a general impression that the cost of a bridge is about or nearly in proportion to its capacity. Such is far from the truth. When the question is considered in detail this is evident. In building a bridge of any span, whether designed for one engine or for another, say 20% heavier, there are the same number of members to handle, practically the same number of holes to punch, ream or drill, and about the same number of rivets to drive in either case. There is practically the same amount of shop work on the lighter structure as there is on the heavier. The erection of the bridge in the field will cost about the same for both cases, as the most rational way of estimating the cost of erection is based on price per linear foot for similar structures. The only item of great importance, in increasing the cost of the heavier bridge, is the first cost of the additional material, and the freight will be increased, though not always, shipments sometimes being made by the car load lot, and the increase of weight for any moderate span would seldom demand an extra car. In connection with this subject, I have gone over the cost of several spans of various lengths recently constructed, in order to determine what the additional cost would have been had they been designed for heavier loads. I find that for plate-girder bridges the increase in first cost for each per cent. increase in capacity is $\frac{1}{4}$ of 1%, and for pin-connected bridges the increase is $\frac{1}{3}$ of 1% for each per cent. increase in capacity. That is, a bridge designed for engine 30 (Mr. Cooper's nomenclature) would only cost from 5% to $6\frac{2}{3}$% more than if designed for engine 25, which is 20% lighter. This is an important fact, and it should be emphasized. For an additional outlay of 5% for plate-girder bridges, or $6\frac{2}{3}$% for pin-connected bridges, an increase of 20% in the capacity of the bridge can be provided for. When this is more generally known and appreciated by our railway engineers and managers, we shall do much to inaugurate an era of stronger bridges on the smaller roads.

PLATE XX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 692.
CHURCHILL ON TRAIN LOADINGS FOR BRIDGES.

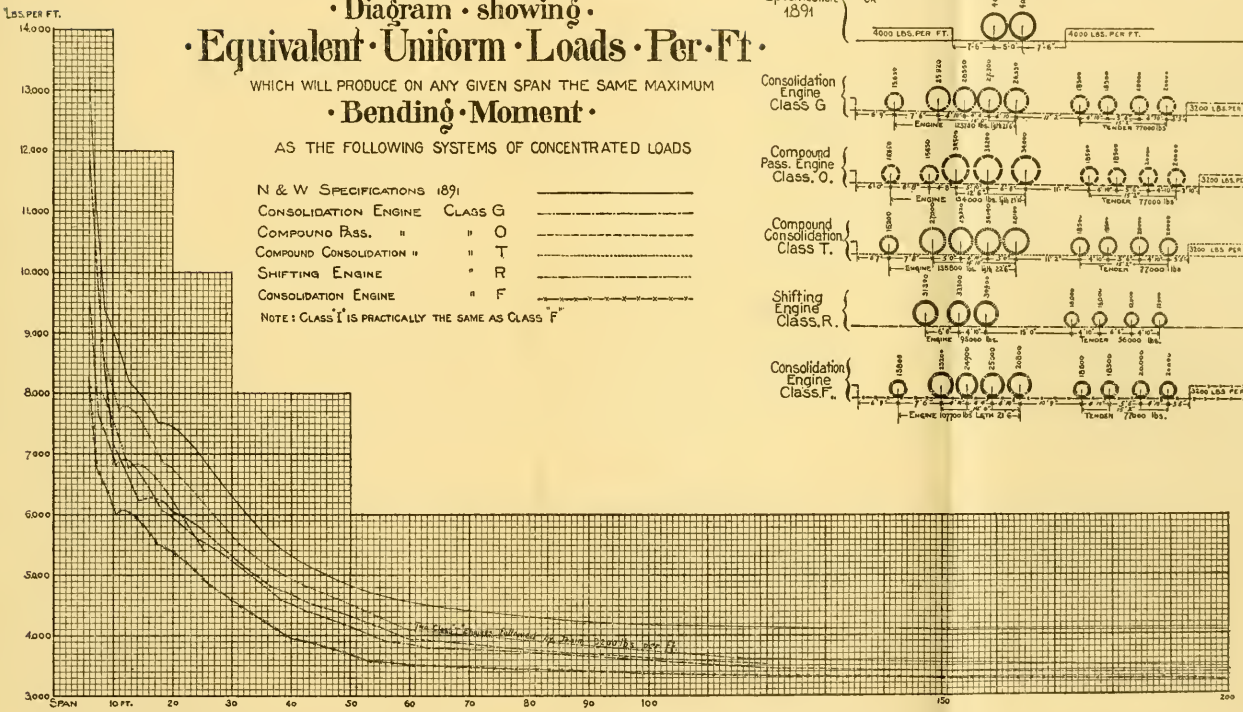
·Norfolk·&·Western·R·R·

· Diagram · showing ·
· Equivalent · Uniform · Loads · Per · Ft ·

WHICH WILL PRODUCE ON ANY GIVEN SPAN THE SAME MAXIMUM

· Bending · Moment ·

AS THE FOLLOWING SYSTEMS OF CONCENTRATED LOADS



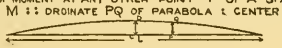
To obtain the maximum bending moment "M" at or near the center of any given span "L", find from the above diagram the equivalent live load "Q" for the given span and loading, add to this "W" the dead load per foot of structure including floor for one track

then, $M = \frac{(Q + W) \times L^2}{8}$

If two girders carry a single track, the moment on each girder is half the above.

For maximum moment at any other point "P" of a span

M : M :: ordinate PQ of parabola : center ordinate CD.



For maximum load on floorbeams spaced a distance l apart use the equivalent uniform load for bending moment on a span $2l$.

Correct: Chas. S. Churchill
Engr Maintenance of Way.

Approved: [signature]
Vice Pres. & Gen. Manager

Office of Eng'r Maint of Way
Roanoke Va: Sept. 1893.

L 90

We are in a transition period from the use of iron to the general use of steel, and we are somewhat in danger of being misled by the results obtained from our specimens in the testing machines as indicative of the strength of the completed structure. We should be liberal in the loads for which we design our bridges, and, on the other hand, we should be conservative in the stresses that are allowed.

Charles S. Churchill, M. Am. Soc. C. E. (by letter).—I heartily agree with the proposition which Mr. Cooper states as follows:

"Throwing aside, therefore, all references to relative merits of actual strains, diagrams, tables or other methods of rapidly approximating to the truth, let us take up and simplify the train loads by reducing them to their simplest forms and least number to cover the necessities of various classes of railroads."

I trust Mr. Cooper will not close this subject till he has reduced the train loads to absolutely their simplest forms. Although his proposed scheme of train loads shows a great increase in uniformity and system over his old specifications, he has not simplified it as far as is possible, as I will proceed to prove.

But, first, as to the weight of freight cars and freight trains of to-day:

The average load per foot of track of one of our regular coal trains is 3 200 lbs.; and we at times have several coal cars coupled together, producing a load of 3 350 lbs. per foot of track, the weight on one axle of a car being 24 500 lbs. Coal cars producing a load of 3 200 lbs. per foot are often transferred from this road to other connecting lines; and there are but very few railroads to-day that do not at times have freight trains averaging for several car lengths 3 200 lbs. per foot.

In view of this fact, I think it is unwise to specify for bridges a following train load of less than 3 500 lbs. per foot. On the other hand, I think 4 000 lbs. per foot is an ample specification loading to meet probable future increase in car loads.

Again, as to engine weights and their train loads, the heaviest engines do not always or even generally haul the heaviest cars. Their special use is to haul longer trains, or to overcome heavy grades on a given division of road; and, *vice versa*, the lightest engines do not haul the lightest cars; therefore, the train load per foot following any engine cannot be said to vary with the weight of the engine. In other words, whatever may be the weights of any engines used by any railroad of standard gauge, the probability is that they will at times haul several consecutive cars, producing a load of from 3 200 lbs. to 4 000 lbs. per foot of track.

I think, therefore, that the following train load is limited to the narrow margin of 3 200 lbs. and 4 000 lbs. per foot.

As to engines less need be said, excepting that a freight engine having four drivers of 28 000 lbs. seems to be about the safe minimum

for a specification, and every road must determine as near as possible what is to be the maximum, both for the entire line and for special divisions.

But now let us examine Mr. Cooper's proposed scheme of loading, taking the heaviest one, "E.-40," as typical of all. He shows two engines coupled, each having drivers of 40 000 lbs. and provided with tenders having axle loads of 24 000 lbs., and then a following train load of 4 000 lbs. per foot. The axle weight of this engine tender is no greater than that on one of our coal cars above referred to, and the load per foot produced by the tender is no greater than the following load of 4 000 lbs.; therefore, why should we not remove the engine tender from our consideration entirely? I made this point and described the practice of the Norfolk and Western Railroad in a previous discussion (see *Transactions*, American Society of Civil Engineers, Vol. XXVI, page 150), and quote the following two paragraphs:

"The advisability of removing the tender wheels of the typical loading was made evident from the fact that the weights on tender wheels are no greater than the weights on wheels of loaded freight cars; and that the increased train load per foot, which it is now found necessary to adopt generally, more than covers any concentration to be found under our tenders. This plan of forming diagrams like the above originated with my principal assistant engineer, Mr. Morgan E. Yeatman. The requirements of the problem at hand speedily showed to us its uses, and brought about the above results. There is no reason why, by the use of such diagrams, a simple type of loading, like that of our specifications, cannot be adopted by every railroad company to suit the requirements of their traffic.

"I trust this discussion may be continued until it results in the entire abandonment of a multiple form of engines for use in bridge specifications; and I hope that the above statement will indicate the uselessness even of the six types of locomotives proposed by Mr. Waddell for this purpose. I would recommend rather the use of a uniform load, with concentrations at not more than four points, which can be so selected as to cover the present and future requirements of the traffic of any railroad. The loading having been adopted, it is a simple matter to prepare diagrams to expedite the practical work of calculation of bridge strains."

I present herewith the Norfolk and Western Railroad Company's standard drawings, showing equivalent uniform loads per foot which will produce on any given span the same maximum bending moment and shearing force as the Norfolk and Western standard specifications of 1891, and several of the heavier types of engines in use at this date on the Norfolk and Western Railroad (Plates XX and XXI).

These diagrams show the engines which the specifications of January, 1891, were prepared to cover in advance of their construction, Class "G" being the heaviest in use at the date of the specification.

It will be noted that these diagrams show not only the effects of the various engines, but the present margin of safety in the specifica-

PLATE XXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 692.
CHURCHILL ON TRAIN LOADINGS FOR BRIDGES.

·Norfolk·&·Western·R·R·

·Diagram·showing·
·Equivalent·Uniform·Loads·Per·Ft·

WHICH WILL PRODUCE ON ANY GIVEN SPAN THE SAME MAXIMUM

·Shearing·Force·

AS THE FOLLOWING SYSTEMS OF CONCENTRATED LOADS

N & W SPECIFICATIONS 1891
CONSOLIDATION ENGINE CLASS G
COMPOUND PASS. " " O
COMPOUND CONSOLIDATION " " T
SHIFTING ENGINE " R
CONSOLIDATION ENGINE " F

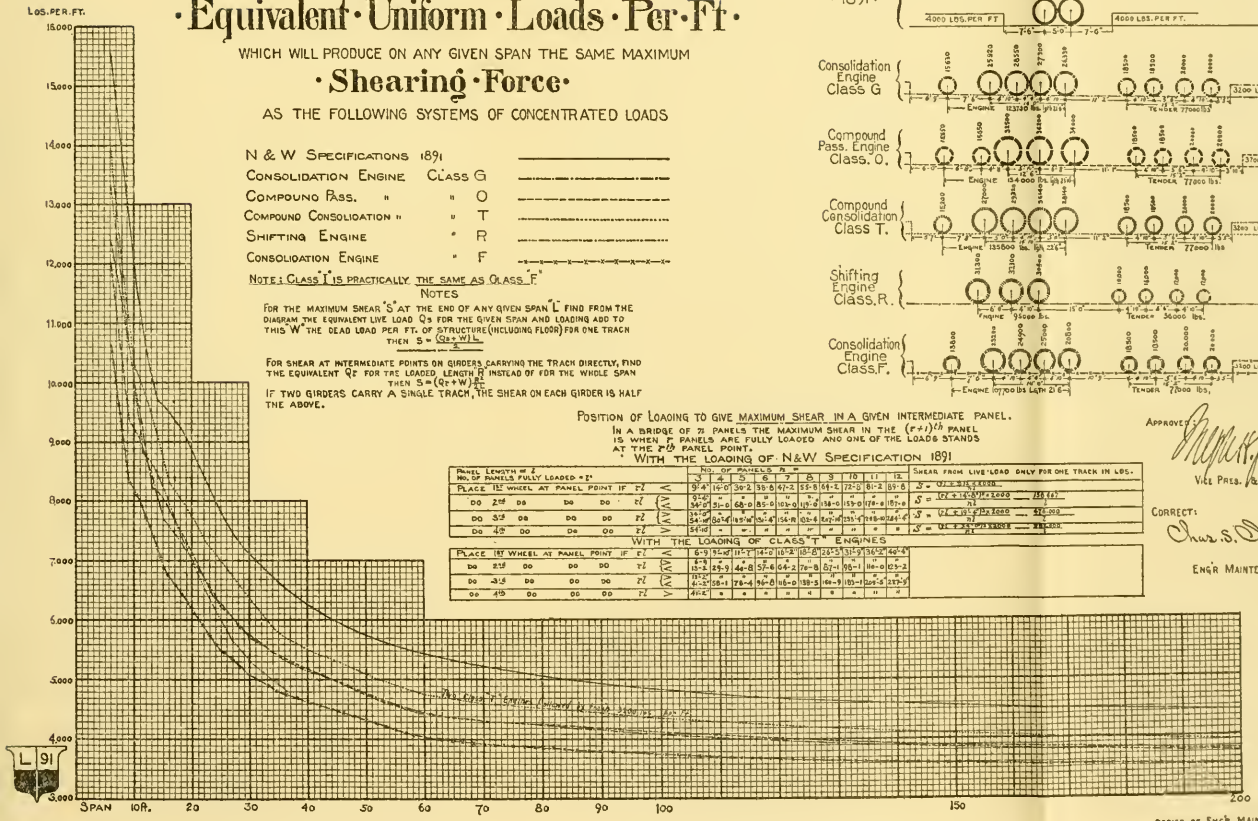
NOTE: CLASS "I" IS PRACTICALLY THE SAME AS CLASS "F"

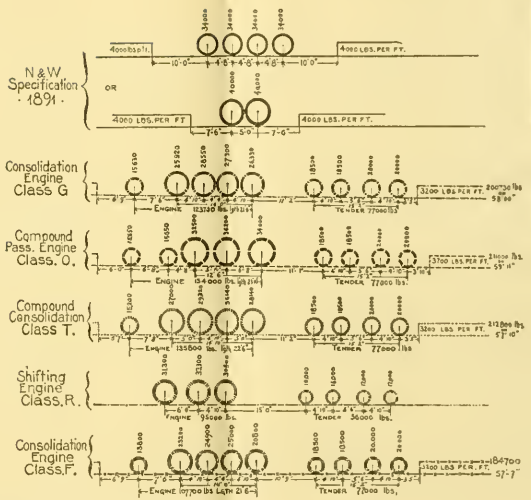
NOTES

FOR THE MAXIMUM SHEAR "S" AT THE END OF ANY GIVEN SPAN "L" FIND FROM THE DIAGRAM THE EQUIVALENT LIVE LOAD Qs FOR THE GIVEN SPAN AND LOADING ADD TO THIS "W" THE DEAD LOAD PER FT. OF STRUCTURE (INCLUDING FLOOR) FOR ONE TRACK THEN $S = \frac{(Qs + W)L}{2}$

FOR SHEAR AT INTERMEDIATE POINTS ON GIRDERS CARRYING THE TRACK DIRECTLY, FIND THE EQUIVALENT Qr FOR THE LOADED LENGTH "R" INSTEAD OF FOR THE WHOLE SPAN THEN $S = (Qr + W)\frac{R}{2}$

IF TWO GIRDERS CARRY A SINGLE TRACK, THE SHEAR ON EACH GIRDER IS HALF THE ABOVE.





POSITION OF LOADING TO GIVE MAXIMUM SHEAR IN A GIVEN INTERMEDIATE PANEL.

IN A BRIDGE OF n PANELS THE MAXIMUM SHEAR IN THE $(r+1)^{th}$ PANEL IS WHEN r PANELS ARE FULLY LOADED AND ONE OF THE LOADS STANDS AT THE rth PANEL POINT.

WITH THE LOADING OF N&W SPECIFICATION 1891

PANEL LENGTH = l / NO. OF PANELS FULLY LOADED = r		NO. OF PANELS n = 3	4	5	6	7	8	9	10	11	12	SHEAR FROM LIVE LOAD ONLY FOR ONE TRACK IN LBS.
PLACE 1ST WHEEL AT PANEL POINT IF rl	<	9'-4"	14'-0"	30-2	38-8	47-2	55-8	64-2	72-8	81-2	89-8	$S = \frac{(r+1+3)l \times 2000}{n}$
DO 2ND DO DO DO rl	≥	9'-4" 34'-0"	" 51'-0"	" 68'-0"	" 85'-0"	" 102'-0"	" 119'-0"	" 136'-0"	" 153'-0"	" 170'-0"	" 187'-0"	$S = \frac{(rl + 14'-8")l \times 2000}{n}$ 138,667
DO 3RD DO DO DO rl	≥	34'-0" 54'-10"	" 80'-4"	" 105'-10"	" 131'-4"	" 156'-10"	" 182'-4"	" 207'-10"	" 233'-4"	" 258'-10"	" 284'-4"	$S = \frac{(rl + 14'-8")l \times 2000}{n}$ 475,000
DO 4TH DO DO DO rl	>	54'-10"	"	"	"	"	"	"	"	"	"	$S = \frac{(rl + 14'-8")l \times 2000}{n}$ 181,000

WITH THE LOADING OF CLASS "T" ENGINES

PLACE 1ST WHEEL AT PANEL POINT IF rl	<	6-9	9'-10"	11'-7"	14'-0"	16'-2"	18'-8"	20'-5"	31'-9"	36'-2"	40'-4"
DO 2ND DO DO DO rl	≥	6-9 13-2	" 29-9	" 44-8	" 57-6	" 64-2	" 76-8	" 87-1	" 98-1	" 110-0	" 123-2
DO 3RD DO DO DO rl	≥	13'-2" 41'-2"	" 58-1	" 76-4	" 96-8	" 116-0	" 138-5	" 160-9	" 183-1	" 204-5	" 227-9
DO 4TH DO DO DO rl	>	41'-2"	"	"	"	"	"	"	"	"	"

APPROVED: [signature]
VICE PRES. & GEN. MANAGER

CORRECT: Chas. S. Churchill
ENG'R MAINTENANCE OF WAY.

OFFICE OF ENG'R MAINT. OF WAY
ROANOKE, VA SEPT 1893

L 91

tion over Class "T" engines. It will be further noted that our very simple form of specification loading produces a load line of practically the same general form as the actual engines and trains. These diagrams alone prove that just such a simple specification loading may be found to cover any engine (actual or typical) desired.

To demonstrate this I have had prepared a similar diagram (Plate XXII) to cover Mr. Cooper's loading, "E.–40." This is shown thus: —— —— ——(3). On this drawing is also shown, for comparison, the diagram for Norfolk and Western Class "T" engine, thus :..... ..... (2); and Norfolk and Western specification loading, thus: ———————— (1); and, as a matter of interest, diagram of Mr. Cooper's loading, "E.–35," is also shown, thus:——*——*——*——*—— (4). This last, it will be noted, very nearly corresponds with the diagram of the Norfolk and Western specifications, but does not follow the diagrams of the actual engines so uniformly.

Now, to substantiate my first statement, there is shown on this same drawing a diagram indicated thus, —— - —— - —— - —— (5), which is almost identical with the diagram of Mr. Cooper's loading, "E.–40" [—— —— —— —— (3)], and the corresponding shear diagrams will also be found to be in close accordance. The loading, however, from which the former was produced is far simpler than Mr. Cooper's "E.–40." Instead of two engines and tenders followed by a uniform load, involving much needless calculation with every bridge, it is simply four loads of 40 000 lbs., 5 ft. apart, replacing a length of 27 ft. of the uniform load of 4 000 lbs. per foot; or, for short spans, an alternative of two loads of 50 000 lbs. 7 ft. apart.

Now, if Mr. Cooper will take this one further step in advance, and get rid of the useless typical engines and tenders, except for a means of reference, using simply their equivalent, which may always be found as above, by not more than four loads uniformly spaced, and replacing some lengths to be determined of this uniform load, he will answer his own proposition and receive the commendation of many railroad and bridge engineers.

I have not in the above discussed the loading "E.–40" in general. It may, however, be of interest to note that the diagram of this load, or my proposed equivalent, gives a slightly greater percentage of excess over trains in actual use on long spans than on short spans. A better typical loading to replace "E.–40" would be four loads of 40 000 lbs. each, spaced 5 ft. apart and replacing 30 ft. of the uniform load of 4 000 lbs., shown by a line thus: —— - - —— - - —— - - —— (6).

It might be well to add that the reason for adopting concentrations at four points instead of a uniform load with a single concentration, as has often been suggested, is that this latter, though it may be so chosen as to represent the equivalent of the actual loading for bend-

ing or for shear separately, will never represent the two correctly at the same time. If chosen as a correct equivalent for bending moments, it will invariably give too low results for shear.

I close by again referring to the last clause quoted above from my previous discussion on this general subject.

JOHN A. FULTON, M. Am. Soc. C. E. (by letter).—The series of engine loads outlined by Mr. Cooper, with his percentage method of variation, ought to afford an excellent assortment to select from; but when we come to consider train loads per linear foot, there appears to be no good reason for assuming that they should bear any specific ratio to the engine loads.

The difference between a freight train on a great through line having light grades, and a freight train on a mountain road having heavy grades, is in the number of cars per train, and not in the loads on individual cars. It would therefore seem that the typical freight train load per linear foot which would be proper for one road should be about the same for any other road.

While any considerable increase in the dimensions of the present box car is not probable, there seems to be no difficulty in the way of increasing the height and carrying capacity of a coal car, although it is now among the heaviest cars in use.

It would be making an important step in the right direction if we could adopt an unvarying freight car load for all standard gauge railroads, and fix its amount at somewhere about 4 000 lbs. per linear foot.

This would be an ideal train load. It would include the effect of many a flat car wheel and would probably be ample for all purposes; but it would seem that anything under 3 000 lbs. per linear foot would be making very little allowance for future possibilities or for present excesses.

It is to be remembered that the average bridge span is not a very long one, that three or four cars will more than cover it, and that in making up trains the yardmen are not supposed to spend much time distributing their heaviest cars through the train in such a way as to keep them separated from each other; and, in consequence, any long train is as likely as not to have its heavy loads bunched in one portion of the train and to produce as ill effects on the bridges as if the whole train were excessive in weight per linear foot. It would also be quite in the line of progress to abandon altogether the practice, now in vogue on many railroad systems, of using a certain engine load for bridges on main lines, and some other and lighter load for bridges on branch lines.

As it is true that a heavy passenger engine may often be sent over an unimportant branch line, so it is also true that a heavy freight engine may often be sent over that same branch line.

The motive power of a railroad is never in charge of the bridge

DIAGRAM SHOWING

EQUIVALENT UNIFORM LOADS PER FOOT

which will produce on any given span the same Maximum

BENDING MOMENT

as the following systems of concentrated loads

N.&W. Specification. 1891 ——— ①

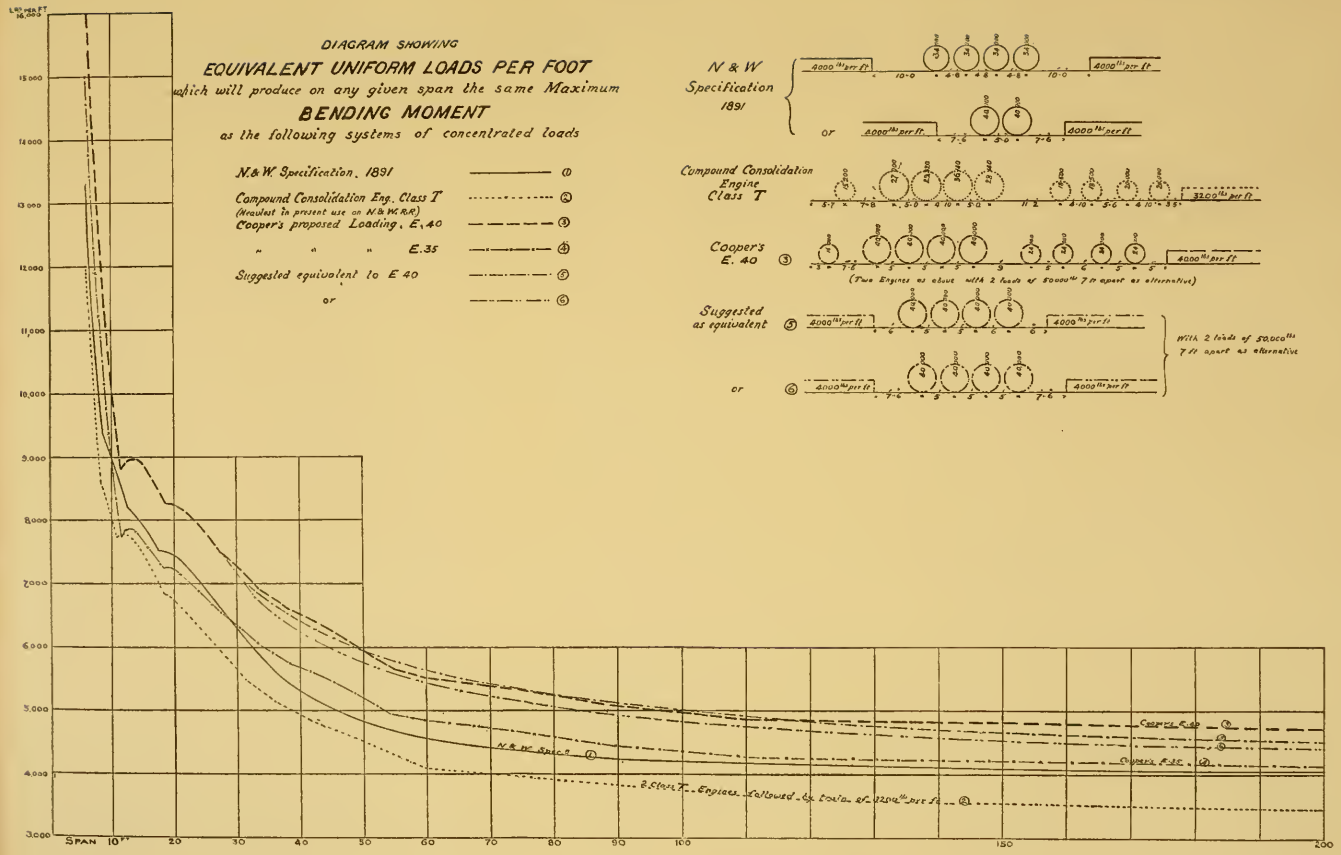
Compound Consolidation Eng. Class T ····· ②
(Heaviest in present use on N.&W.R.R.)

Cooper's proposed Loading. E.40 — — — ③

" " " E.35 —·—·— ④

Suggested equivalent to E 40 —·—·—·— ⑤

or —·—·—·— ⑥



department, and when the master mechanic has a call for an engine he sends out the one that seems to him the most available, regardless of its effects on the bridges.

EDWIN THACHER, M. Am. Soc. C. E. (by letter).—Of all plans which have been proposed for simplifying train loadings on railroad bridges, the writer considers the plan proposed in this paper by far the simplest and best, and he can see no reason why it cannot be consistently and profitably adopted by every railroad company in the United States. The wheel spacings and the distribution of load on the different wheels appear to be well chosen, so far as the writer can judge, except that he would consider a distance of 8 ft. between axles for the concentrated loads safe; this, however, is of little consequence, as these loads apply only to very short spans.

As it is desirable to know, without trial, which of the two loads will produce a maximum moment, the following formulas are given:

Let W = weight on one driver concentrated load,
" W = " " " freight engine,
" L = length of span in feet,
" M = bending moment in foot-pounds,—then

Concentrated load 1 driver on span $M = W\frac{L}{4}$ ..... (1)

" 2 " " $M = \frac{W}{2}\frac{(L-3.5)^2}{L}$ ..... (2)

Freight engine 3 " " $M = W\left(\frac{3}{4}L - 5\right)$ .... (3)

" " 4 " " $M = W\left[\frac{(L-2.5)^2}{L} - 5\right]$ (4)

Equating (1) and (2) $L = 11.95$ ft.
" (2) and (4) $L = 18.66$ " hence the following:
Concentrated loads $\begin{cases} L < 11.95 \text{ use (1)} \\ L > 11.95 \text{ " (2)} \end{cases}$
Freight engine $\begin{cases} L < 18.66 \text{ " (3)} \\ L > 18.66 \text{ " (4)} \end{cases}$
For load "E.–25" $W = 20\,000$, $W = 12\,500$
Equating (2) and (4) $L = 22.78$.
For load "E.–40" $W = 25\,000$, $W = 20\,000$,
Equating (1) and (3) $L = 11.43$.

As the concentrated loads do not apply to spans exceeding 22.78 ft. for the lighter engine, or to spans exceeding 11.43 ft. for the heavier engine, they are practically confined to the calculation of track stringers of moderate length, and cause no complication in bridge stresses.

WARD BALDWIN, M. Am. Soc. C. E. (by letter).—I have read Mr. Cooper's paper with great interest on account of the valuable statis-

tics it contains, and also because the same plan Mr. Cooper suggests for arranging a set of bridge loadings, viz., grading them so that the parts of each loading have a uniform ratio to the corresponding parts of any other loading of the set, was adopted by the writer in a set of loadings he sent, in January, 1893, to J. A. L. Waddell, M. Am. Soc. C. E., as a suggested modification of Mr. Waddell's proposed set of standard loadings. If a set of loadings, such as Mr. Waddell has proposed, meets all requirements, then the advantages of such a modification seem to be sufficient to warrant its adoption ; and it might be of interest to hear from Mr. Waddell at this time why he rejected this suggestion. If present practice, however, may be accepted as a fair criterion of the general limits a set of standard loadings should cover, the conclusion seems inevitable that a set of loadings graded so that the uniform load bears a uniform ratio to the driver load will not include all the live loads that ought to be specified. With the purpose of ascertaining what conditions a set of standard loadings should fulfill, the writer secured copies of the bridge specifications in force in 1893 on 100 independent railway systems in this country, and made a comparative study of the loadings therein. A full list of these loadings and a proposed set of standards were given by the writer in a discussion of the paper by C. D. Purdon, M. Am. Soc. C. E., on "Comparison of Modern Engine Loadings, with Standard Specifications for Spans from 10 to 200 Feet," read at the Chicago Congress. A glance at this list of loadings will make it plain that in the specifications now in use the uniform load is not even approximately proportional to the engine loading.

For instance: Twenty-one specifications unite a 30 000-lb. driver load to a 3 000-lb. uniform load, and 14 unite the same driver load to a 4 000-lb. uniform load.

Again: The Pennsylvania Railroad unites a 30 000-lb. driver load to a 4 000-lb. uniform load; while the Atchison, Topeka and Santa Fé Railroad unites a 44 000-lb. driver load to a 3 200-lb. uniform load. Unless it is true that local conditions do not require the coupling of heavy engines to light trains, and *vice versa*, then the set of loadings proposed by Mr. Cooper would not appear to be comprehensive enough. Mr. Cooper's statement that "there is far greater probability of harmony on this subject than on many other disputed points in practice" encourages the writer to reiterate the hope that this question will be referred to a committee of the Society.

J. C. Bland, M. Am. Soc. C. E. (by letter).—In Mr. Cooper's paper, "Train Loadings for Railroad Bridges," the writer thinks there is a means of arriving at an agreement as to a series of standard loadings, especially so as the author has distinctly eliminated the already much-discussed question as to which of several methods is best to follow in obtaining stresses. The subject to which the author has confined him-

self is largely a question of fact and of familiarity with the several kinds of rolling stock in general use; and the matter for discussion is then on the possibility of representing by a base loading two consolidation engines and their appropriate car loads; whether the latter can be expressed by a constant ratio to the engine driver loads, and whether the tenders and truck axle loads can likewise be expressed by another constant ratio. Both of these questions involve some difficulty, for, with practically the same engines, one road demands greater tender capacity (coal and water) than another, due entirely to differences in physical conditions, and by no means connected with character of car loading. Some variations in the ratio of tender to driver axle loads are shown in Table No. 1 (page 196).

Another point which somewhat complicates the question is, whether, in a series of standard loadings, a constant wheel base spacing can be maintained for the engines as the driver loads increase in weight? An examination of data of actual engines shows a fairly constant rigid wheel base of engine, and also of tender, *per se* but an increase of from 20 to 30% in the space between rear driver and first tender axle as the engine becomes heavier; in short, the engine length increases with the weight. For some data regarding this see Table No. 1. In the opinion of the writer neither question is of serious importance, for, in the case of the tender capacity, a development of water and coal stations, at any time likely to take place on a given line, reduces the objection, though the tank capacity remains; and as regards the increased wheel base for the heavier engines, it only at the most, amounts to 2½ or 3 ft., or an increase of, say, 5% in total engine length.

The other question to settle is, having a given consolidation engine, what train load is it proper to consider as following it? Mr. Cooper considers the appropriate train one which weighs per linear foot one-tenth the weight on the driver axles. The writer has worked for some time on this subject, but from a somewhat different starting point, not the weight per foot of train, but the weight per car axle, desiring to express the latter by a constant ratio to load on driver axle. He, therefore, by so doing, had also to fix on a standard car length and axle spacing, and in consequence of selecting the hopper-bottom coal car as a type of spacing, gets somewhat heavier car loads per linear foot to accompany a given engine than Mr. Cooper, viz., about 11% greater. Mr. Cooper's remarks, on pages 179 and 180, together with his tables, leave little to be said on the question of car storage capacity. The 4 000-lb. train load, which he places as a maximum, gives on his assumed car length of 37 ft. a total of 148 000 lbs., that is, 37 000 lbs. on each of four axles. The writer having taken—corresponding to a 40 000-lb. driver load—a car axle load of five-sixths, that is, 33 333 lbs., gets, by reason of the assumed car length of 30 ft.,

TABLE No. 1.

Data on Consolidation Engines.

Name.	Date.	Weight in Working Order.						Tender.		Cylinder.	Wheel Base.				
		On truck.	On drivers.	On engine.	On tender.	Total engine and tenders.	Ratio tender to drivers.	Water galls.	Coal, lbs.		Engine. Rigid	Engine. Total.	Between engine and tender.	Tender.	Total engine and tender.
West Shore	1883	16 000	88 000	104 000	64 000	168 000	0.727	3 000	10 000	20″ x 24″	14′ 0″	21′ 7″	10′ 4″	15′ 8″	47′ 7″
Class. I, Penna. R. R.	1876	12 240	79 400	91 640	55 770	147 340	0.692	3 000	8 000	20″ x 24″	13′ 8″	21′ 6″	10′ 9″	15′ 4″	47′ 7″
Class I, Penna R R.	1886	13 000	82 700	95 700	57 800	153 500	0.699	3 000	8 000	20″ x 24″	13′ 8″	21′ 6″	10′ 9″	15′ 4″	47′ 7″
Class. R. Penna. R. R.	1886	14 025	100 600	114 625	57 800	172 425	0.574	3 000	8 000	20″ x 24″	13′ 10″	21′ 9″	11′ 8″	15′ 4″	48′ 9″
Class. 112, D. & R. G. R R.	1888	16 700	95 800	112 500	68 000	180 500	0.710	.....	.....	20″ x 24″	13′ 6″	21′ 4″	11′ 0″	16′ 0″	48′ 4″
Class. 1134, R. G. W. Rwy.	1891	16 700	98 300	115 000	68 000	183 000	0.691	3 200	15 680	20″ x 24″	13′ 6″	21′ 6″	11′ 0″	15′ 10″	48′ 4″
Colorado Midland.	1888	12 000	114 000	126 000	82 000	208 000	0.719	4 000	16 800	20″ x 26″	14′ 6″	22′ 0″	10′ 6″	15′ 6″	48′ 0″
Baldwin "Octopod"	1890	13 700	133 800	147 500	82 800	230 300	0.619	.....	.....	.....	14′ 0″	22′ 3″	12′ 1½″	14′ 5″	48′ 9½″
Reading, Class. I, 75	1888	14 700	138 334	153 054	71 980	225 014	0.520	.....	.....	.....	14′ 0″	22′ 3″	13′ 3½″	15′ 8″	51′ 2½″
Reading, Class. I, 55	1888	12 500	110 300	122 800	64 140	186 940	0.581	.....	.....	.....	13′ 8″	21′ 9″	12′ 3½″	15′ 7″	49′ 7½″
Reading, Class. I, 53	1888	15 380	104 120	119 500	64 140	183 640	0.616	.....	.....	.....	13′ 8″	21′ 9″	12′ 3½″	15′ 7″	49′ 7½″
Reading, Class. I, 50	1888	15 380	101 470	116 850	64 140	180 990	0.632	.....	.....	.....	13′ 8″	21′ 9″	12′ 3½″	15′ 7″	49′ 7½″
Baldwin, 10-34 E., 1 271 to 1 290	1891	16 000	96 000	112 000	74 000	186 000	0.772	.....	.....	.....	13′ 6″	21′ 4″	13′ 4″	16′ 0″	50′ 8″
Western N. Y. & P. Compound No. 175	1891	13 050	116 550	129 600	.....	.....	.....	.....	.....	.....	14′ 0″	21′ 9″			
Western N. Y. & P. Compound No. 169	1891	16 220	110 650	126 870	.....	.....	.....	.....	.....	19″ x 26″	14′ 0″	21′ 9″			
Ills. Central	1892	18 700	118 600	137 300	.....	.....	.....	3 850	.....	.....	16′ 9″	24′ 5″			

a deduced load per linear foot of 4 444 lbs., that is, an 11% greater load per linear foot from a 10% less axle load. The ordinary freight train is a medley of various sorts, kinds and lengths of cars from all over the country—box, stock, flat and coal cars mixed; except as regards the large house cars of western lines, there seems no limit to the interchange of car stock. The writer has seen Lehigh Valley coal cars in the middle of Utah. Excluding coal or ore carrying roads, on the average line, it is only at certain seasons of the year that one sees continuous trains of the heavier class of freight, viz., coal, ore and the like.

Interchange of engines, being comparatively rare, may be considered as inadmissible, though now and again, as Mr. Cooper points out, passenger engines travel over other lines than their own; and for this reason the writer considers, no matter what the freight traffic may be, that it is prudent to design the bridges for two such concentrated axle loads as has been Mr. Cooper's practice, especially as it only affects the short spans. Corresponding to the loading, Class E.-30, of Mr. Cooper, the writer has used two concentrated axle loads of 50 000 lbs. each, spaced 8 ft. apart, which recently he changed to 7½ ft. Though there is little opportunity for increase in the widths or heights of cars in the East, in the West, while the widths would remain fixed, there is room for increase in height, as the standard headroom above rail is greater. The Santa Fé requires 22 ft., the Denver and Rio Grande Railroad and Rio Grande Western Railway require 21 ft. The highest freight car which the writer has seen is one 13 ft. 6 ins. from top of rail to top of running board, to which, adding 6 ft. for a man, we get 19 ft. 6 ins., which leaves with above headrooms, at least 1½ ft. for increase; and as such a car has about 8¾ to 9 ft. clear inside height, such height can be increased at least 15%; and, therefore, the storage capacity by that percentage. Though this be the fact, the writer doubts its utility, for to load such a storage space with freight of the heavier class would, by the increase of axle loads, demand heavier rails than are customary in the West. The rule seems to be, the longer and larger the car, the lighter its axle load, for such cars are designed only for the lighter freight.

The writer for a long time having had the hope of being able to express the spacing of the axle loads of engines, tenders and cars in terms of the distance between the drivers, and also the loads on each axle in terms of the load on each driving axle, had collected and tabulated all the data obtainable regarding the rolling stock of several first-class railways, noting at the same time the characteristics concerning gradients, curvature and the nature of traffic. It was thought that all axle spacings might be found—with such a degree of approximation as seemed necessary or desirable—to be multiples of 2 or of 2½ ft., and also that the ratio of tender load to driver load would be close to two-thirds. The data tabulated comprised that for 8-wheel, 10-wheel and consolidation engines, a later page of the latter being given

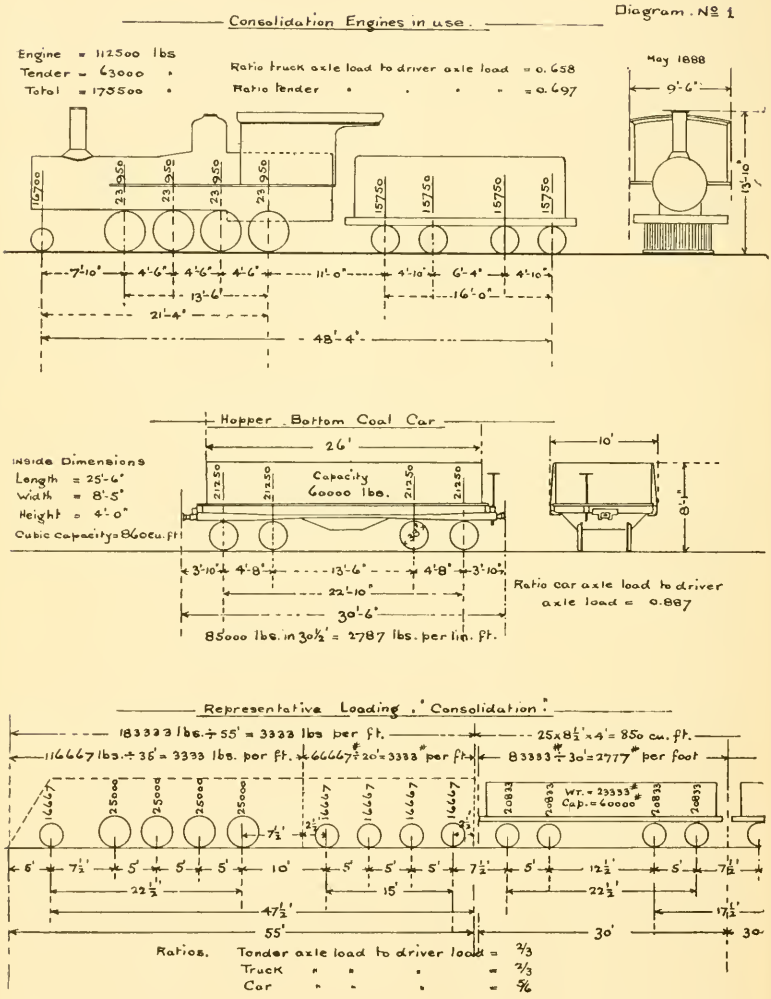
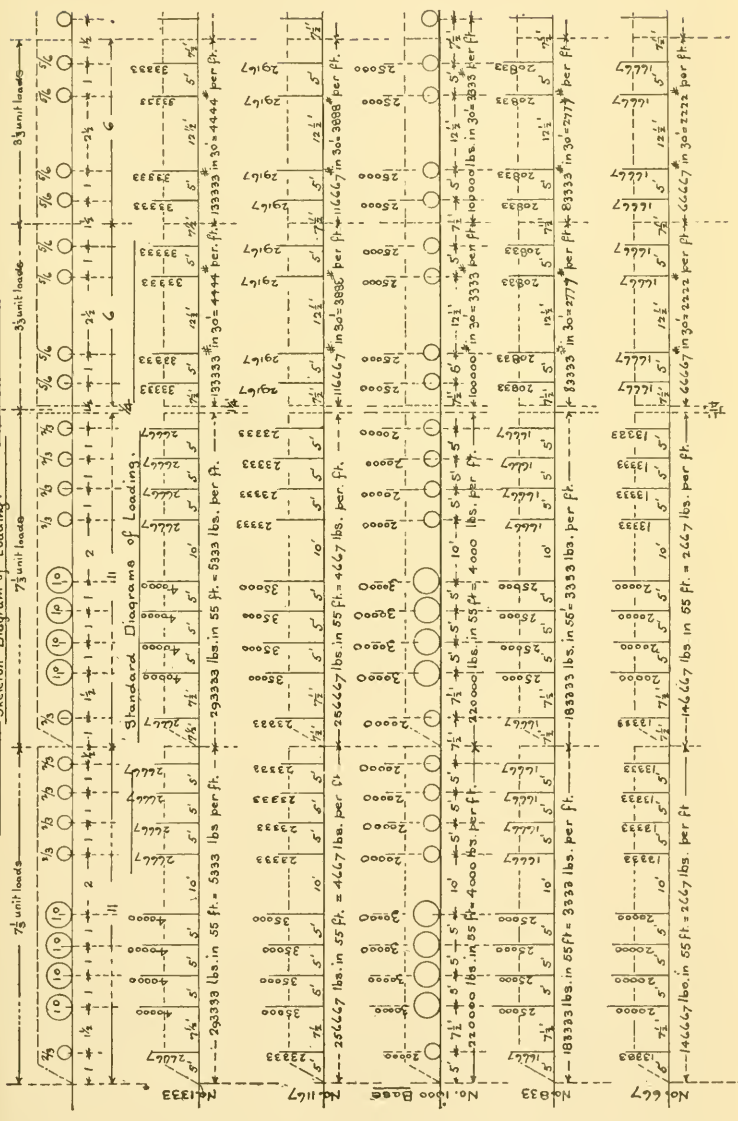

Consolidation Engines in use.
Diagram. Nº 1
Engine = 112500 lbs
Tender = 63000 "
Total = 175500 "
Ratio truck axle load to driver axle load = 0.658
Ratio tender " " " " = 0.697
May 1888
9'-6"
13'-10"
15700
23950
15750
7'-10"
4'-6"
4½"
4'-6"
11'-0"
4'-10"
6'-4"
13'-6"
16'-0"
21'-4"
48'-4"
Hopper Bottom Coal Car
Inside Dimensions
Length = 25'-6"
Width = 8'-5"
Height = 4'-0"
Cubic capacity = 860 cu. ft
26'
Capacity
60000 lbs.
21250
10'
8'-1"
3'-10"
4'-8"
13'-6"
22'-10"
30'-6"
Ratio car axle load to driver axle load = 0.887
85000 lbs. in 30½' = 2787 lbs. per lin. ft.
Representative Loading. "Consolidation."
183333 lbs. ÷ 55' = 3333 lbs per ft.
116667 lbs. ÷ 35' = 3333 lbs. per ft.
66667 ÷ 20' = 3333 # per ft.
25 x 8½' x 4' = 850 cu. ft.
83333 # ÷ 30' = 2777 # per foot
16667
25000
20833
WT. = 23333 #
Cap. = 60000 #
6'
7½"
5'
10'
22½'
15'
47½'
55'
12½'
17½'
30'
Ratios. Tender axle load to driver load = ⅔
Truck " " " " = ⅔
Car " " " " = ⅝

Diagram N° 2.


'Skeleton' Diagram of Loading.
Standard Diagrams of Loading.
No. 1333
No. 1167
No. 1000 Base
No. 833
No. 667

in Table No. 1 (page 196). From this table the writer concluded that a standard spacing for consolidation engines could be as shown on the "Representative Loading," foot of Diagram No. 1 (page 198), wherein it is seen that all distances are multiples of $2\frac{1}{2}$ ft.

Regarding the axle loading, it was concluded that, taking the driver weight as unity, the loads on the truck axle and on each tender axle could be represented by the fraction $\frac{2}{3}$. That this fraction was in general too large in case of the truck axle load was realized; but inasmuch as it was the writer's practice to disregard the truck load in determining the shears, though including it in computing the cumulative moments and the reactions, it was allowed to stand. Whence the weight of a consolidation engine was taken to be $4\frac{2}{3}$ units, the tender to be $2\frac{2}{3}$ units, and the total weight of engine and tender to be $7\frac{1}{3}$ units, the unit being the load on each driver axle.

As to the spacing, the engine wheel base was taken $4\frac{1}{2}$ units, the tender wheel base 3 units, the distance from engine to tender 2 units, the total wheel base of engine and tender $9\frac{1}{2}$ units, and total length of engine and tender to be 11 units, the unit spacing being the distance between the driving-wheel axles. A "Skeleton Load Diagram" being then formed (see Diagram No. 2, page 199), the writer computed the cumulative moments at each axle in terms of the driver axle load as unity and driver axle spacing as unity. From this table of "skeleton moments" the moments for any consolidation engine could readily be found by multiplying the tabular "skeleton" moment by the product of the driver axle load and driver axle spacing; or rather the "skeleton" stresses were determined in any instance, and then multiplied by the foregoing product for the stresses due to engine in question. For example: If 5 ft. be taken as the spacing of the driver axles, then the factor by which to multiply the "skeleton" moments is five times the driver axle load; or if, as some have maintained, we should adhere to $4\frac{1}{2}$ ft. as the driver distance, then the factor would become four and one-half times the driver axle load. In the first instance the actual spacing would be as shown on the five standard load diagrams of Diagram No. 2 (page 199); in the second instance the spacings would be 90%; *i. e.*, the wheel distances would reduce respectively to 4.5 ft., 6.75 ft., 3 at 4.5 ft., 9.0 ft., 3 at 4.5 ft., 2.25 ft. = 49.50 ft. total length. For a narrow-gauge engine, the driver distance could be taken 4 ft., whence the following spacing, 4.0 ft., 6.0 ft., 3 at 4.0 ft., 8.0 ft., 3 at 4.0 ft. 2.0 ft., = 44.0 ft. total length; that is, 80% of the spacing on the Standard Load Diagrams, which agrees closely with existing narrow-gauge engines.

An examination of the tabulated car stock data collected indicated considerable variation in loading and spacing of cars of the same capacity and type, which the writer concluded was largely due to the precedent and peculiarities ruling on each road; he therefore

concluded to await an opportunity to secure data of recent car stock where such restrictions did not obtain. Such an opportunity presented itself about three years ago when new stock was being purchased by a road with the characteristics of which the writer was familiar; and there was this further advantage, the motive power was bought about the same time and was designed by the Superintendent of Motive Power and Machinery, who was not only thoroughly versed in all modern practice, but was also particularly well informed as to the requirements of his own line. Table No. 2 gives the car

TABLE No. 2.

STANDARD GAUGE FREIGHT CARS, D. & R. G. R. R.

	Coal Cars.		Low flat.	Box car.	Stock car.
	Gondola.	Hopper-bottom.			
Outside dimensions—Length 0 to 0 B. N.	34′ 6″	30′ 6″	34′ 8½″	34′ 6″	34′ 6″
" " Width 0 to 0	9′ 8″	10′ 0″	9′ 2″	9′ 3″	9′ 3″
" " Height 0 to 0	7′ 7½″	8′ 1″	3′ 4″	11′ 6″	11′ 6″
Inside dimensions—Length	30′ 0″	25′ 6″	32′ 0″	31′ 5¾″	31′ 5″
" " Width	8′ 2″	8′ 5″	8′ 10″	8′ 1¾″	8′ 1″
" " Height	3′ 6″	4′ 0″	.....	6′ 1½″	6′ 1½″
" " Cubic contents	858	860	.....	1 568	1 556
" " Cross-section	28.6	33.7	.....	49 9	49.5
" " Floor area	245	215	283	256	254
Wheel base—Truck	4′ 8″	4′ 8″	4′ 8″	4′ 8″	4′ 8″
" Total	27′ 2″	22′ 10″	27′ 2″	27′ 2″	27′ 2″
Weights—Dead weight	24 000	25 000	21 000	26 000	26 000
" Nominal loading	60 000	60 000	60 000	60 000	60 000
" Total	84 000	85 000	81 000	86 000	86 000
Deductions—Dead weight per foot of length	696	820	605	754	754
" Nominal loading of length	1 739	1 967	1 729	1 739	1 739
" Total " "	2 435	2 787	2 334	2 493	2 493
" Dead weight on each axle	6 000	6 250	5 250	6 500	6 500
" Live " "	15 000	15 000	15 000	15 000	15 000
" Total load "	21 000	21 250	20 250	21 500	21 500
" Cubic capacity per foot of length	24.9	28.2	.....	45.5	45.1
" " " height	245	215	.....	256	254
" Nominal loading÷Cubic capacity	70	70	.....	38	39
" Dead weight÷Nominal loading.	0.4	0.417	0.35	0.433	0.433
" " ÷Total loading	0.286	0.294	0.259	0.302	0.302
" Nominal loading÷Dead weight.	2.5	2.4	2.86	2.31	2.31

stock referred to, the consolidation engine in use being shown at head of Diagram No. 1 (page 198).

The writer considered a representative trainload to be one composed of a series of hopper bottom coal cars, a diagram of one of which is shown underneath the consolidation engine, and from these two he formed a "Representative Load Diagram," two consolidation engines heading a train of hopper-bottom coal cars (see Diagram No. 1, page 198), in which the spacing throughout is a multiple of 2½ ft., the loading on truck and each tender axle two-thirds the load on driver axle, and

the load on each car axle five-sixths the load on each driver axle. This loading being representative of the actual traffic on the line, for proportioning the bridges, it was considered prudent to increase it 20%, whence the "typical load diagram" is shown on Diagram No. 2 (page 199), where it is called "No. 1 000 Base."

From this typical load diagram as a base (viz., two 220 000-lb. engines heading a train of coal cars, each being 100 000 lbs. in 30 ft.), a series was constructed, each one varying from the base load by one-sixth and one-third, above and below, diagrams of which are shown in Diagram No. 2, numbered above the base 1 167 and 1 333, and below the base by 883 and 667, the numbers being indicative of the ratios to the base load.

Here the writer would express his appreciation of Mr. Cooper's singularly simple notation, indicating, as it does, at once the load on the driver axles and the car load per linear foot possible by reason of his car load being one-tenth the load on each driver axle. Such a notation would be inapplicable in the foregoing series of loadings, wherein the deduced car load per foot is 11% the driver axle load, since the foundation of the base diagram is the car axle load, and not the load per linear foot. The series of typical loadings shown on Diagram No. 2 were framed for the writer's own use, and he hardly expected to find application for No. 1 333 or No. 667, as he considered Nos. 1 167, 1 000 and 833 representative of good practice in their respective places, that loading to be used which was 20% heavier than the weight of the actual trains in operation.

The writer draws a distinction between a "representative loading" and a "typical loading," meaning by the former a loading fairly close to the actual weights in use, but with their irregularities in loads and spaces rounded off or reduced to some uniformity; that is, the "representative load" is the one which truly and fairly represents the heaviest trains on the line. By the typical loading is meant one for which the bridging is designed, a load non-existing on the line, but to which limit it is supposed the actual or representative loading will tend, and in time reach.

The writer, in giving an account of his own work in the direction of a standard loading, does not thereby offer its results in opposition to Mr. Cooper's proposed loadings (from which it differs mainly in the train loadings), though he considers it rational to express the car load by axle concentrations when the engine load is so expressed. Though believing the ratio of tender to driver weight to be fairly represented by the fraction two-thirds, the variations in existing engines are too great to warrant objection to Mr. Cooper's ratio of six-tenths. And while deeming Mr. Cooper's 9 ft. between rear driver and first tender axle not borne out by the run of existing examples, and in the case of the heavier engines being from 25 to 30% too short, yet, as this is to

the advantage of the bridge, such lack of accordance with the actual machine is of no moment whatever.

The suggestion which the writer offers, however, is that Mr. Cooper's two concentrated weights be made uniform for the whole specification, viz.: Two loads, each 50 000 lbs. and spaced 7½ ft. apart, or 50% farther apart than the driver axle loads, for heavy passenger engines, of which such weights are representative, are as likely, by interchange, to run over a line whose bridges are of the class E.-25 as one whose bridges are of the class E.-35 or E.-40. The writer, not desiring to bring into his paper anything on those questions which Mr. Cooper in his article has expressly excluded from discussion, is, however, of opinion, based on experience, that bridge loading expressed by concentrated weights and definite spacing, extending even to the car load, is more appreciated and more intelligible to general managers and chief engineers than any equivalent uniformly distributed loading, or uniformly distributed load plus a concentrated weight, though the writer admits he strongly favors and supports the latter method in the present diversity, which exists in the several well-known specifications of railroads and bridge companies. The average general manager cares nothing about modes or methods of calculation; he is interested only in results. In building his bridges he merely wants to be assured by good authority that they are properly proportioned to carry the assumed loading.

The desire for unity forms the quest of philosophy. Reducing the whole field of bridge engineering to a common standard (and therefore a dead level) is not, however, unity of the kind sought. This sort of unity would change the engineer into a calculating machine; he is already quite enough of that. In engineering there should be no such thing as "Rome has spoken—it is settled." The writer holds it not only possible but likely that fully 25% of that which now forms the subject-matter of many "received" specifications will suffer material change at the hands of the coming generation of bridge engineers, even though the clauses in mind are now worded with all the certainty and "cocksureness" of a set of 39 articles, or of the utterances of certain professors of physical science and biology when sailing on foreign seas.

Theodore Cooper, M. Am. Soc. C. E.—I have listened to the discussion this evening with much interest, but feel that some of the writers have failed to comprehend my position. I take the stand that we must have a standard train loading for our specifications, with which we can approach the managers of railroads when we wish to inform them in regard to the character of the bridge we propose to build for them. It will be a pictorial representation of what the bridge is to be capable of doing. In no other manner can we convey to the average railroad man the capacity of the structure; tables, diagrams,

concentrated loads, etc., will not do it. Of course, where railroads have their own bridge engineers, the selection of such a typical loading as I propose may not be necessary. Such engineers should be in direct touch with the management, and have their confidence. They can proceed in their own way to carry out the desired purpose. But I presume that the larger percentage of railroads in the United States build their bridges without the advisory action of a specially qualified bridge engineer. They simply ask the bridge companies for proposals upon some accepted specification, selecting with more or less judgment the kind of train load they desired used. It is for this class that general specifications are intended. I am not trying to convert the experienced engineer to depart from any methods of calculation which he may have determined to be sufficiently correct for his purposes. But it must be noticed that, in defending his methods, he is always anxious to prove that it only varies from certain standard train loads by certain small percentages, thus inferentially recognizing the necessity for a standard of final reference, an actual train load.

My specifications are in pretty general use throughout the United States, and I believe one feature that has been a satisfactory one is the series of engines from which any manager can select that engine best suited to his purpose. In preparing for a new edition I am willing to adopt a new series of train loadings, if we can settle upon a series that will be more generally accepted. The ones proposed in my paper appear to be far simpler than any that have been elsewhere proposed. The criticism made that they differ in certain features from any actual engines is true; but as these are intended for representative types solely for the purposes of proportioning our bridges, it is of no moment. It would be impossible to select a properly proportioned engine for the minimum size and then increase by regular gradations, as is proposed, without getting disproportionate results for the higher engines. It is necessary then, if we desire to retain the principle of increasing by percentages, to make a compromise in the proportional weights between the light and heavy engines. The weights on the pony wheels could be made less and those on the tender wheels more if found desirable, but the results would not be much different. As to the criticism that there is no necessary connection between the weight of the engine and the weight of the following train, this is correct; but, nevertheless, the tendency will always be in the direction of using heavy engines upon roads handling heavy cars. The great mass of bridges will receive their heaviest loads from two engines (all spans less than 120 ft.), whether the train is one car or fifty.

The remarks of Mr. Blakeley are very much to the point, and I am glad to have him bring out this point of the small increase of cost by the adoption of a heavier class of engine. It was my intention to have added some data in this direction, but I concluded it would be better if

brought out by an engineer of a bridge-building firm. It would have more weight with the purchasers of bridges to receive such a statement from the actual builders of such work than from an engineer. If this point was more generally appreciated, there would be fewer bridges built to the lighter specifications. What manager would hesitate to increase the strength of a 125-ft. span 20% for an additional cost of only $200? I hope the tendency of the discussion will be towards wiping out the lighter engines now used in our specifications. Personally I should like to exclude all engines and trains less than E.-30 of my paper.

J. E. Greiner, M. Am. Soc. C. E. (by letter).—In the present discussion it is hardly necessary for us to inquire very closely into the various causes which led up to the existing unreasonable conditions of train loads specified by railroads and bridge engineers, but in addition to the reasons advanced by the author we could perhaps add that the personal vanities of engineers have favored just such diverse conditions.

A maker of specifications tries to be original, or at least to be thought so, and therefore changes the usual permissible stresses, rearranges the clauses in some standard specifications, inserts a cranky idea of his own here and there, and designs an entirely new typical engine.

Whether or not the author can succeed in having a set of standard loads, such as proposed, generally adopted, will depend almost entirely upon the stand taken by those engineers who have already outlined a set of standards for themselves. They must be offered something as good or better than they now have, and must also be convinced of the advantages of such general standards.

We, undoubtedly, all agree with the author's general statements that refinements in dimensions or weights are useless, that the freight train loads are the only car loads necessary to consider, and that two types of engines can be made to represent the loadings produced by passenger and freight trains.

We must also admit that since train loads assumed for calculations of stresses are but typical, and that a change from the class now in use to a similar class with a slightly different distribution of loading and spacing would make practically no difference on our stress sheets.

The proposed types are very simple, agree closely with actual engines, and embrace the variation in weight which should meet all cases. The gradual increase in the weight of the loads by a definite percentage is a particularly good feature. This same plan has been followed on the Baltimore and Ohio Railroad during the last ten years, the 86-ton engines and tenders of 1884 having been increased first 10%, then 20%, and finally 25 per cent. Train loads increased in the same proportion.

The specified loading during the last five years, up to nine months ago, was 107.5-ton engine and tender, followed by 3 750 lbs. per foot. There are engines and tenders running over the road which weigh 107 tons. The specified loading was therefore recently increased to 125-ton engines and tenders, followed by 4 000 lbs. per foot. The wheel spacing and loading of this last typical engine are different from any engine known to the writer, and also bear no direct proportion to the old 86-ton typical engine; but having been used in the design of important bridges on the road, the writer did not think it worth while to make any change, and has therefore adopted this engine in the 1894 specifications, although he is not responsible for its design.

Should an understanding in regard to the author's proposed standards be reached, the writer, for one, will lend hearty co-operation, and would be perfectly willing to change his specifications accordingly.

The life of an iron railroad bridge has heretofore been scarcely 25 years, not because the bridge wore out under the traffic for which it was designed, but because the loading during that period has increased to such an extent that the older bridges, being generally of poor design, were no longer to be trusted. In 1869 the first 10-wheel camel engine, weighing 90 700 lbs., was run over the B. & O. R. R. In 1892, or about 23 years later, consolidation engines weighing 134 000 lbs. were brought into service. This shows an increase of 47.7% during the period. How much longer this increase will continue is a question on which railroad men differ. That there is a limit none can deny, and that this limit has not yet been reached is maintained by the best railroad men in the country.

Economical transportation is the great aim of railroad corporations; but as freight rates, through competition and other causes, are continually being forced down, and wages endeavoring to hold their place, something must be done outside of cutting wages to meet the downward tendency of rates. It is thought that this difficulty will be overcome by increasing the motive power, so that more tons of freight can be handled by the same train crew. So long as this will offset the low rates, we may expect the increase to be advocated; but we must bear in mind that heavier motive power means increased cost of maintenance, and that, after it has passed a certain limit, bridges which were perfectly safe under lighter loads will have to be strengthened or renewed at large outlays. The strength of existing bridges will, therefore, in many cases, determine the limit of the weight of our engines, as it is reasonable to suppose that the slightly reduced cost of transportation will hardly warrant the renewal of many bridges. There is no question as to this limit on the older railroads, but on new roads it will depend entirely on the economy of transportation.

Mr. Cooper's proposed E.-40 weighs 176 000 lbs. without tender. It

will probably be as heavy or heavier than any which will ever come into general use, but in the writer's opinion it would be a great waste to design structures for loads so much greater than are in service at the present time. A bridge built during the last five years by an intelligent and experienced designer, using proper materials and having first-class workmanship, should stand indefinitely under a train service even in excess of what it was figured for. In fact, the writer ventures to state that when the upward limit of engine weights becomes once definitely fixed, engineers will greatly increase their permissible stresses. Bridges are being designed for assumed typical engines, heavier than those in service; but even with this increase in view, we know that we have given the structure a large margin over and above what is necessary for our assumption.

So far as the train loading is concerned, the writer fully agrees with Mr. Cooper in the opinion that 4 000 lbs. per foot will not be exceeded except in isolated cases. In fact, the heaviest train load on many roads does not, and probably never will, exceed 3 000 or 3 200 lbs. per foot, except occasionally, and bridges which are perfectly safe for such loads will not be injured by an occasional heavier train load.

The writer can see no more necessity for proportioning all bridges, on branches as well as main lines, for the heaviest passenger engines than for the heaviest freight engines. Even if the heaviest engines should occasionally run over the branches, our bridges would be fully capable of carrying them. A large number of trains passing over a structure will wear it out sooner than heavier loads passing at longer intervals, as must be known to all railroad engineers having charge of bridges.

The writer knows of a number of bridges built at the same shops, during the same year and from the same plans, where one was placed on the main line and all the others on a branch. The bridge on the main line was racked to such an extent that it had to be trestled after 12 years' service. The others show no effects of wear, and are, to all appearances, just as good as when first built. The number of trains passing over the main line was probably three or four times greater than of those passing over the branch, although the weights of the engines were practically the same in both cases. It seems to the writer that, while such instances as these decidedly point out that bridges on main lines should be designed for the heaviest engines, they do not indicate any such necessity for branches having from one-quarter to one-third the traffic.

Joseph M. Wilson, M. Am. Soc. C. E. (by letter).—I quite agree with Mr. Cooper in his general remarks on the subject in question. To develop a certain rigid standard specification for the designing of bridges, complete in all its details, fixing absolutely by rule every item in the fundamental data, and then requiring certain exact methods to

be followed for the determination of every part of the structure, reduces the engineer to a mere machine which might as well be operated, as it were, from a central station by playing on a keyboard, and all originality, professional judgment and progress are lost.

It unfortunately happens that the system now almost universally adopted in this country, of having manufacturers prepare their own designs with their own engineers, has necessitated limiting the freedom of the engineer by minute specifications, which would not be the case if the engineer who designed the structure were an officer of the railroad company. The liberty of the engineer is therefore already more curtailed than wholesome for originality and progress. Under any circumstances, however, it is well in certain matters by "fair and thoughtful discussion" to approach uniformity of action, and Mr. Cooper's proposal to take up one subject at a time is a good one.

Perhaps no one is more important than train loading. The great difficulty with our older bridges was that not enough allowance was made in this item to allow for the rapid development and increase of loading which has taken place during the last 10 or 20 years; and while I am of the opinion, with Mr. Cooper, that the practical limit of loading has probably been nearly reached, still, I am in favor of a liberal allowance in this respect.

One of the important advantages of stone bridges lies in the fact of the great excess of strength over theoretical requirements, which, from the nature of the material and mode of construction, cannot be decreased. What I would like to see is a little of this in our metallic bridges.

Great refinements in dimensions and loading may, under such circumstances, well be omitted, as, for any material advance in the future, it is possible that the whole detailed distribution of engine loading might be changed, and an exact system adopted now would not suit then.

So far as the concentrated wheel method of computing bridges is concerned, it evolved very naturally in my own practice. The original rule followed in Pennsylvania Railroad practice was to use a live load of 2 000 lbs. per linear foot of track, which was in the year 1860 increased to 3 000 lbs. No account was taken of engine loading. About 1871–72, notice began to be taken of the increased concentrated loading on girders and other short spans from the driving wheels of engines, and a system of calculation was adopted providing for this, the concentrated wheel method being then practically started. In addition to this, the constantly increasing weights of engines of various types placed in service on the road and its branches required opinions from the Engineering Department from time to time as to the capacity of the bridges on these several lines to sustain such service. The question was not that of computing new bridges, where an assumed load

might be adopted, ample to cover all supposed traffic for years in advance, but of an opinion in reference to certain existing engines, and it became necessary to work up the exact conditions of the problem, a problem of constant occurrence on long railroad lines in active operation.

When a decision must be given on the reliability of an actual structure in use, under an established service, the bridge being perhaps already quite up to, or beyond, the limit of what might be called the standard, the concentrated wheel method must, in my opinion, still be used, and no general assumption will properly reach the case.

It naturally follows that, after using such a method for computing existing structures, it was adopted for new bridges also.

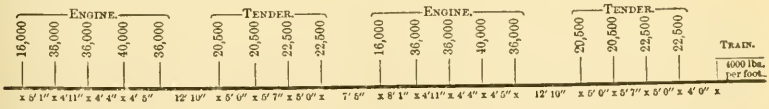
Then it was that typical engines were outlined, as representing the greatest development which would probably occur for some years ahead, and these were submitted to the Motive Power Department of the road for approval, and accepted.

In accordance with the practice at that time on the Pennsylvania Railroad, complete details of all bridges were prepared in the Bridge Department of the company, and no specifications were required for manufacturers, beyond those for material and workmanship. It was, therefore, some years before complete printed specifications were issued, giving methods of computation, the first being about 1882, and in the meantime an actual engine, Class M, had been put into service, the effects of which, under some conditions, were in excess of those from the typical engines, and this explains why, after such typical engines had been designed, a third engine was placed on the list.

It was supposed that ample allowance had been made for future increase for a long time to come, but actual engines now in use are already exceeding the loads of the typical engines then adopted.

The loading for our (Wilson Brothers & Co.) revised specifications of April 11th, 1891, used on the Philadelphia and Reading Terminal Railroad, is as follows:

Two typical engines coupled, weighing, with tender, 125 tons each, distributed as follows:



connected with a train weighing 4 000 lbs. per linear foot of track.

The "basis for discussion" offered by Mr. Cooper is certainly in the right direction, and while I am hardly prepared to say just now what modification, if any, I should propose in it, it meets with my approval in its general principles, and I should be glad to see substantial results follow its consideration.

HENRY B. SEAMAN, M. Am. Soc. C. E. (by letter).—In discussing Mr. Waddell's paper on "Some Disputed Points in Railway Bridge Designing," I ventured to suggest (*Transactions*, Vol. XXVI, p. 228), that the maximum concentration be 40 000 lbs., that the concentrations under the tender should be all alike, and should not be less than 23 000 lbs. each, and that the pony wheel of the engine should not exceed 16 000 lbs.

The engines outlined by Mr. Cooper seem very near the ideal, and his general proposition of change by percentage is an admirable one. It is to be regretted, however, that he found it necessary to use fractions of a foot in his wheel spacing, and in this respect the spacing suggested by Mr. Waddell seems preferable. The distance of 9 ft. between engine and tender is considerably more than that used in former typical engines, and is more than is often found in practice. The present tendency of new engines, however, is to increase this distance, and sometimes even to exceed that shown in the diagram. The drivers, on the other hand, are, in some instances, 4 ft. 6 ins. apart.

In adopting typical engines for new construction, we should not forget that they are not intended for examination of existing structures.

We are indebted to Mr. Cooper for this step toward a uniform practice, and particularly so since his discussions have not appeared to favor a movement of this kind.

WILLIAM BARCLAY PARSONS, M. Am. Soc. C. E. (by letter).—This paper marks a long step in advance in the matter of what shall constitute bridge loads, and, coming from Mr. Cooper, will probably be accepted more generally and more readily than if it had been written by any other advocate of the wheel load system. The great advantages that the proposed loadings offer are their great simplicity, freedom from small and senseless fractions of dimensions and a systematic increase in weights. But this simplicity openly abandons, in a large degree, the idea that the strains obtained by using wheel loads conform closely to the strains actually produced in practice.

The prime argument, therefore, urged against the use of a distributed load, with or without computations, that such a method gave results not strictly in accordance with those obtained by the use of wheel loads, can be urged with equal force against the loads suggested in this paper, which, too, will give strains differing from those found by using real or previously accepted typical engines. Of course, such accuracy is quite unattainable, and, even if it were, would be sadly modified by irregularity of construction, variations in adjustment, condition of track, inaccuracy of engine balance, and a host of other practical considerations. The difference, therefore, that separates those who, like myself, believe in equivalent distributed loads, and those who prefer wheel loads, has by this paper been much reduced.

In his verbal discussion of his own paper, Mr. Cooper gave an excellent reason for keeping typical engines in specifications, which was that managing non-professional officers pass on bridge construction who do not understand what an equivalent distributed load means. It may be necessary to keep up the typical engines for them, but that is not a reason for forcing them upon engineers.

If the corresponding equivalent distributed loads were computed and put in the specifications jointly with these typical engines, with their percentage of increase as proposed, an excellent and rational system would be obtained. Instead, however, of using two large axle loads, it might be well to use three, or a mogul design, to be employed in designing spans up to 20 ft. above which the distributed load is to apply.

Though I quite agree with Mr. Cooper that a standard specification is quite impossible and most undesirable, still I do think that a scale of bridge loadings to be generally adopted is possible, and Mr. Cooper by his paper has gone more than half way toward a ground where all can meet.

T. Kennard Thomson, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Cooper's paper cannot fail to please all bridge engineers and renew their hopes that a few standard engine diagrams will be adopted.

If the Society would appoint a committee to suggest two or three, or, at the outside, six, standard engine diagrams, I am confident that the majority of our railroads would at once make their selections therefrom. While our Society seems rather averse to making such suggestions, it would by so doing confer a great boon on many of its members.

As has been said, this question has been unfortunately confused with various methods of making the subsequent calculations. For while an engine diagram is as plain as a picture to the average railroad official, a table of varying uniform loads per foot is quite incomprehensible to many; and, besides, our grievance is not against the engine diagram pure and simple, but against the aggravating number and variation of them.

But if a few standards were selected, then each individual engineer could select what short cuts he desired, and, for that matter, complete tables of bending moments, shears, equivalent uniform loads, etc., would speedily be published and at the disposal of all. As an engineer who has been employed by both railroads and bridge builders, I endorse what Mr. Blakeley has said about our first-class railroads being willing to pay a few dollars more for the sake of getting a very much better bridge, which is only true economy. This, unfortunately, cannot be said of some of our second-class builders.

David L. Barnes, M. Am. Soc. C. E. (by letter).—Mr. Cooper's

paper calls attention to the very important fact that the dimensions of freight cars are nearly at a maximum at the present time, owing to the vertical and lateral dimensions having reached the limits permissible within the right of way, and he argues that this fact gives rise to another of equal importance, namely, that the limit of weight per running foot of freight train will be practically reached when freight cars having the present maximum cross-section are loaded full with heavy grain. All that Mr. Cooper has said about the limits of weights and dimensions is easily shown to be true from an examination of the conditions under which freight cars are handled in this country. The practice of interchange of cars between different roads and the use of cars for a large variety of freight both tend toward the final adoption of a standard car that can be used, with reasonable economy, for both heavy, through and light local freight. Some enthusiasts who have traveled in foreign countries have been greatly impressed with what appeared to be a discovery of an error in American railroad practice, namely, the use of too large and too heavy cars for freight; but on making estimate of the effect of the use of the four-wheel light cars so common in Europe, it was found that the cars would be out of service most of the time and would be hauled light more often than loaded.

The next step in the reduction of the cost of car repairs and inspection must come from the adoption of a standard car that will be nearly the same in all particulars on all roads. This will so reduce the cost of repairs and facilitate the interchange of cars that most of the disputes in settlement and delays in transfer, now so common, will disappear. Steel under frames and steel in freight car bodies will not be generally adopted until there is some approach to a standard size, for the reason that the longer life of a steel freight car framing would prolong its existence to a period when standards will surely be adopted, and then it would be a great inconvenience to have undersized cars that are too good to be thrown away. The extra expenditure, about $100, for a metal car framing to get greater durability will not be made so long as there is no harmony in dimensions and details. The standard car will probably be made in two lengths, 36 and 40 ft., and will probably have about 75 sq. ft. of cross-section. Its capacity for a uniform distributed load will be about 100 000 to 120 000 lbs., with an empty weight of about 35 000 to 40 000 lbs. The flat and gondola cars will probably have the same under frames and will weigh somewhat less, as such cars have no roofs. Any one not familiar with the labors and trials of car inspectors and repairers at interchange points, can hardly realize the vast saving of labor that would result from the introduction of a uniform type of car details.

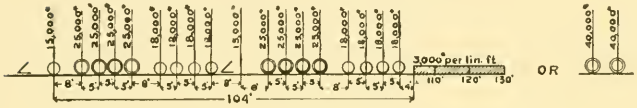
With locomotives as well as with cars the economical limit of weight will soon be reached. At present it requires a skilled man to fire the

THE COMPROMISE STANDARD SYSTEM OF LIVE LOADS FOR RAILWAY BRIDGES.

Sheet No. 1.

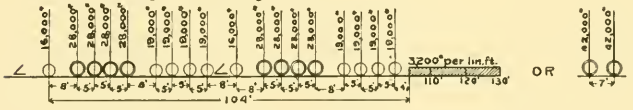
CLASS Z.

Total Weight of One Engine and Tender=93.5 Tons



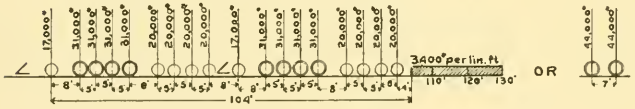
CLASS Y.

Total Weight of One Engine and Tender=102 Tons.



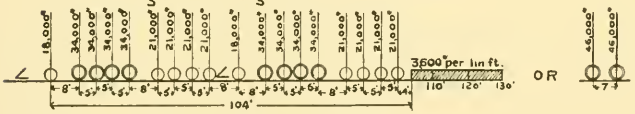
CLASS X.

Total Weight of One Engine and Tender=110.5 Tons.



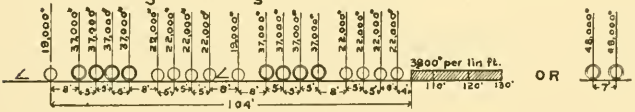
CLASS W.

Total Weight of One Engine and Tender= 119 Tons.



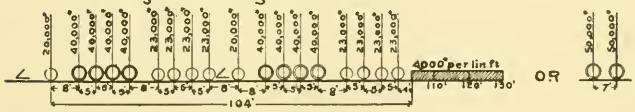
CLASS V.

Total Weight of One Engine and Tender= 127.5 Tons.



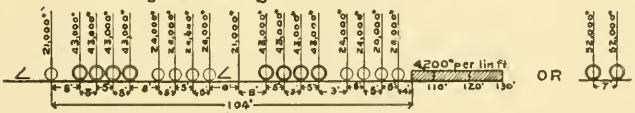
CLASS U.

Total Weight of One Engine and Tender. 136 Tons.



CLASS T.

Total Weight of One Engine and Tender = 144.5 Tons.



N.B. These concentrated loads are to be employed only as a standard of reference, and are the loads which were used in obtaining the data for plotting the curves on sheets 2, 3, & 4. For finding stresses use the equivalent loads given by said curves.

10 ft.-6 in. and 11-ft. fireboxes of large engines, and in the West, where the old type of deep, short and narrow fireboxes has been in use for a long time, the firemen grumble about the large grates. To fire a large grate a man must actually see the fire and place his coal where it is needed, while with the deep fireboxes the fuel rattles into place somewhat naturally. The principal economy of heavy engines and large cars is derived from the heavy trains of reasonable length that they permit; this gives a considerable reduction of labor per ton mile. But with the use of the quick-action air brake, which reduces the number of trainmen required, and the lower rates of labor that are clearly approaching, there will be less to be saved by hauling larger train loads than can be hauled with the present maximum weight of locomotives. The possible saving in fuel will be more attractive and remunerative, and the result will be a reduction of the rate of fuel consumption from 150 lbs. per square foot of grate per hour, the present average, to 90 or 100 lbs., which will give a large money saving, especially where coal is dear, as has been found in Mexico, where wages are low and fuel is high.

Altogether this paper by Mr. Cooper is very suggestive and points to one of the most productive fields of search for greater national economy, that of uniformity in design of such structures as are in common use.

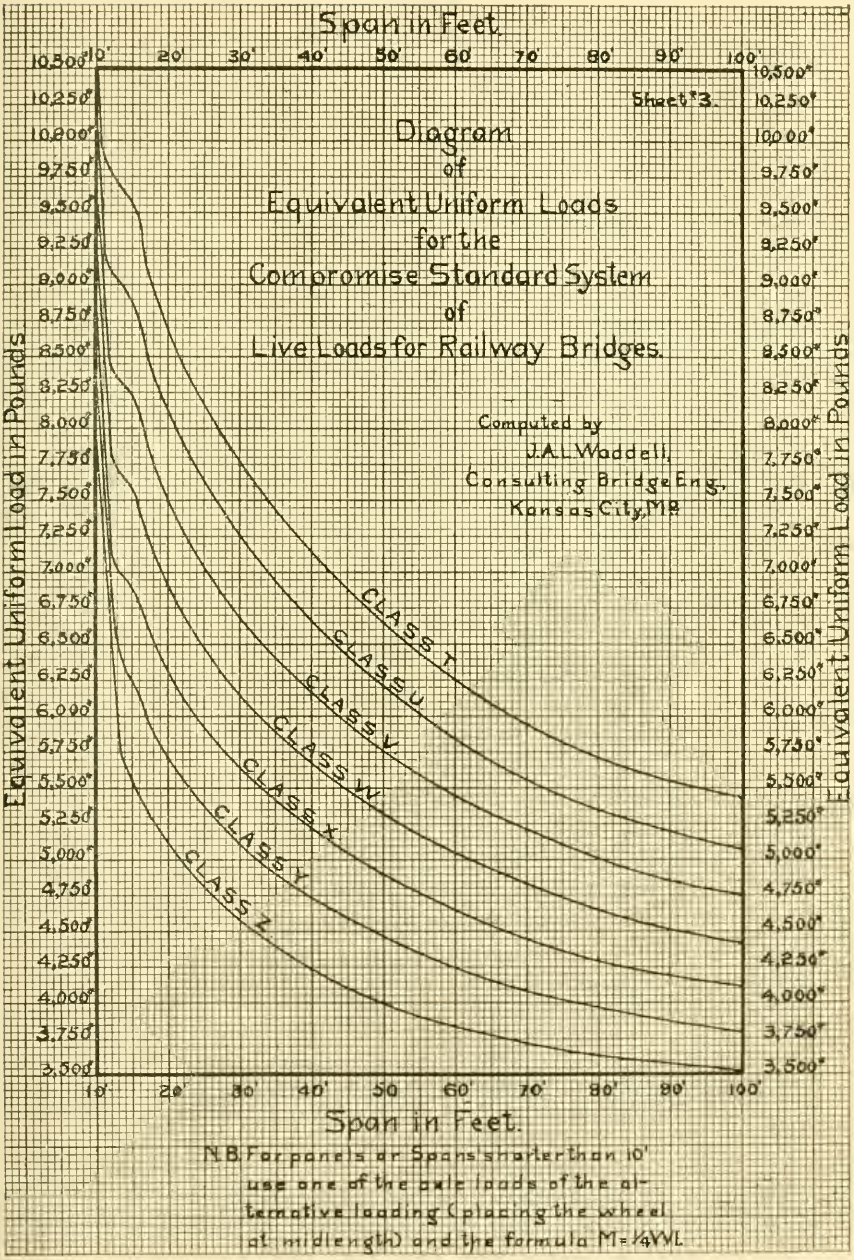
J. A. L. Waddell, M. Am. Soc. C. E. (by letter).—In discussing Mr. Cooper's paper there are two or three points upon which I wish to touch. The first is in respect to the advisability of establishing a uniformity in methods of bridge designing. While it is impracticable and fundamentally unadvisable to limit the designer in every petty detail, in that it would destroy individuality and retard progress, it is, on the other hand, greatly to be desired that some kind of system be established out of the chaos in which bridge designers are now laboring. Thus, for instance, if standard live loads, proper intensities of working stresses for all portions of bridges, the actual effects of impact, a correct practical column formula for steel compression members determined by experiments on full-sized struts, the best quality of materials from both an engineering and an economic standpoint, a standard for testing, and a standard of excellence for shopwork, were established, no possible harm could be done to the engineering profession; but on the contrary great good would result. Nor need the establishment of these standards be absolutely final; for they could be made, so to speak, to a certain extent elastic, so as to conform to future changes in loading of trains, improvements in manufacture of metal, etc. The tendency of the times is towards the systemization of technical knowledge; and, in my opinion, it is the duty of each member of our profession to do his share to aid in such systemization.

In treating of standard live loads for railway bridges at this late date, Mr. Cooper is a little behind the times; for nearly three years ago I brought the subject to the attention of the members of the American Society of Civil Engineers in a paper entitled "Some Disputed Points in Railway Bridge Designing," one of the principal of said points being the question of standard live loads. This paper was very thoroughly discussed by the members of the Society; and the discussion of the live load question has been continued, until a few months ago, in the technical press and by correspondence, the result of the entire discussion being a consensus of opinion, from which I have prepared and published in pamphlet form a "Compromise Standard System of Live Loads for Railway Bridges and the Equivalents for Same," and have distributed it very generally among those railway and bridge engineers who are, or who ought to be, interested in the subject, including every chief engineer of a railroad in North America whose address could be found.

This "Compromise System" differs materially from Mr. Cooper's proposed system in several particulars, as can be seen by referring to Plate XXIII; for while his heavy train load agrees very closely with "Class U" of the "Compromise System," his lightest loading differs essentially from that of "Class Z," in that his weights on tender axles and his car loads per linear foot are much smaller. By referring to Plate XVI, opposite page 90 of my paper on "Some Disputed Points," etc., it will be seen that the tender axle loads for my first proposed "Class Z" were 15 000 lbs., the same as Mr. Cooper has assumed for his lightest loading. Now, as it was due to the general consensus of opinion that I increased said tender loads to 18 000 lbs. per axle, it is evident that in this particular Mr. Cooper's lightest loading will not meet with the approval of the engineering profession.

Again, very few engineers can be found to-day who would countenance the use of a car load for bridges less than 3 000 lbs. per linear foot, knowing that it is possible to load some cars, as Mr. Cooper shows, as high as 4 000 lbs. per linear foot; consequently, Mr. Cooper's lightest car load of 2 500 lbs. per linear foot is decidedly too light.

Again, as it is generally conceded that the heaviest train loading should be about the same as that of "Class U" of the "Compromise Standard System," or that assumed by Mr. Cooper as his maximum; and as it is generally conceded that the lightest train loading should consist of two engines like those of the "Compromise Class Z" (or Mr. Cooper's proposed lightest engines, with heavier tender loads), followed by a car load of 3 000 lbs. per linear foot, it results that Mr. Cooper's proposed "multiple" or "proportional" system of live loads, although very pretty in theory, will not meet the requirements of practical men, especially general managers and chief engineers of railroads.


Span in Feet.
Sheet 3.
Diagram
of
Equivalent Uniform Loads
for the
Compromise Standard System
of
Live Loads for Railway Bridges.
Computed by
J.A.L. Waddell,
Consulting Bridge Eng.
Kansas City, Mo.
CLASS T
CLASS U
CLASS V
CLASS W
CLASS X
CLASS Y
CLASS Z
Equivalent Uniform Load in Pounds
Span in Feet.
N.B. For panels or Spans shorter than 10' use one of the axle loads of the alternative loading (placing the wheel at midlength) and the formula M = 1/4 WL

This "multiple" or "proportional" system, however, is not a new idea, for a year or more ago the same suggestion was made to me by a well-known professor of civil engineering. It was with regret that I declined to adopt it, my sole reason being that it would not meet the practical requirements of the case; because, while axle loads for modern engines vary from 25 000 to 40 000 lbs., maximum car loads per linear foot for the various railroads vary only from 3 000 to 4 000 lbs.

But, after all, does the proposed "Multiple System" possess any real advantage over the "Compromise Standard System"? An inspection of Plates XXIV and XXV will answer this question in the negative; for the curves thereon for all spans, excepting such short ones as are seldom adopted, vary from one another by regular gradations; consequently for any span length the equivalent live loads per linear foot increase in arithmetical progression, and therefore the weights of metal per linear foot for the various classes do approximately the same. The percentages of increase, however, for equivalent load and weight of metal per linear foot vary with the length of span. For instance, referring to Plate XXV, we find the following percentages of increase in load from one class to the next in reference to the loadings for "Class Z :"

100-ft. span	9.44%
200-ft. span	9.06%
300-ft. span	8.51%
400-ft. span	8.06%
500-ft. span	7.70%

With the diagrams of equivalent loads at hand, an engineer can make his interpolations and his "multiples" just as readily with the "Compromise Standard System" as he could with Mr. Cooper's proposed "Multiple System."

Just here it appears to me proper to say a few words more in respect to the "Compromise Standard System" and the "Equivalent Uniform Load Method," for the reason that both subjects were first brought up before the American Society of Civil Engineers in a paper presented thereto for discussion, and afterwards the discussion was continued by correspondence (subsequently published by the technical press), confined to the members of that Society, directly or indirectly interested in bridge building, until a general consensus of opinion was obtained.

After ascertaining that nine-tenths of the members of the Society interested in the subjects favored both the "Compromise Standard System" and the "Equivalent Uniform Load Method," some three or four months ago I addressed a circular to all the chief engineers of

railroads in North America whose addresses I could obtain, with a request that they vote on the following ballot:

BALLOT

On the Use of the "Compromise Standard System of Live Loads for Railway Bridges" and the "Equivalent Uniform Load Method."

I $\frac{\text{AGREE}}{\text{DO NOT AGREE}}$ to use the "Compromise Standard System of Live Loads for Railway Bridges" when calling for bids on railway bridge work, or when having plans prepared for railway bridges.

I $\frac{\text{AGREE}}{\text{DO NOT AGREE}}$ to specify that the "Equivalent Uniform Load Method" is to be used in computing stresses in the bridges that are to be designed for my road.

Signature of voter.

.....

Chief Engineer of the

.....

.....

(Please fill out the name or names of your road or roads.)

Please cancel with ink either the words "DO NOT AGREE" or the word "AGREE," so that the remaining words shall indicate your intention.

Up to date, not quite one-half of the parties addressed have been heard from, and the ballots continue to come in slowly; consequently, I shall not be able to announce the result of the ballot in the technical press for probably several months.

Of those heard from, about 4% objected to voting; 6% pleaded illness, want of time, etc.; 6% voted in the negative; 6% divided their votes, favoring either one of the questions, but not being prepared to vote on the other; 9% favored both questions, but did not wish to make any formal agreement, and 69% gave an unqualified vote in favor of using both the "Compromise Standard" and the "Equivalent Uniform Load Method."

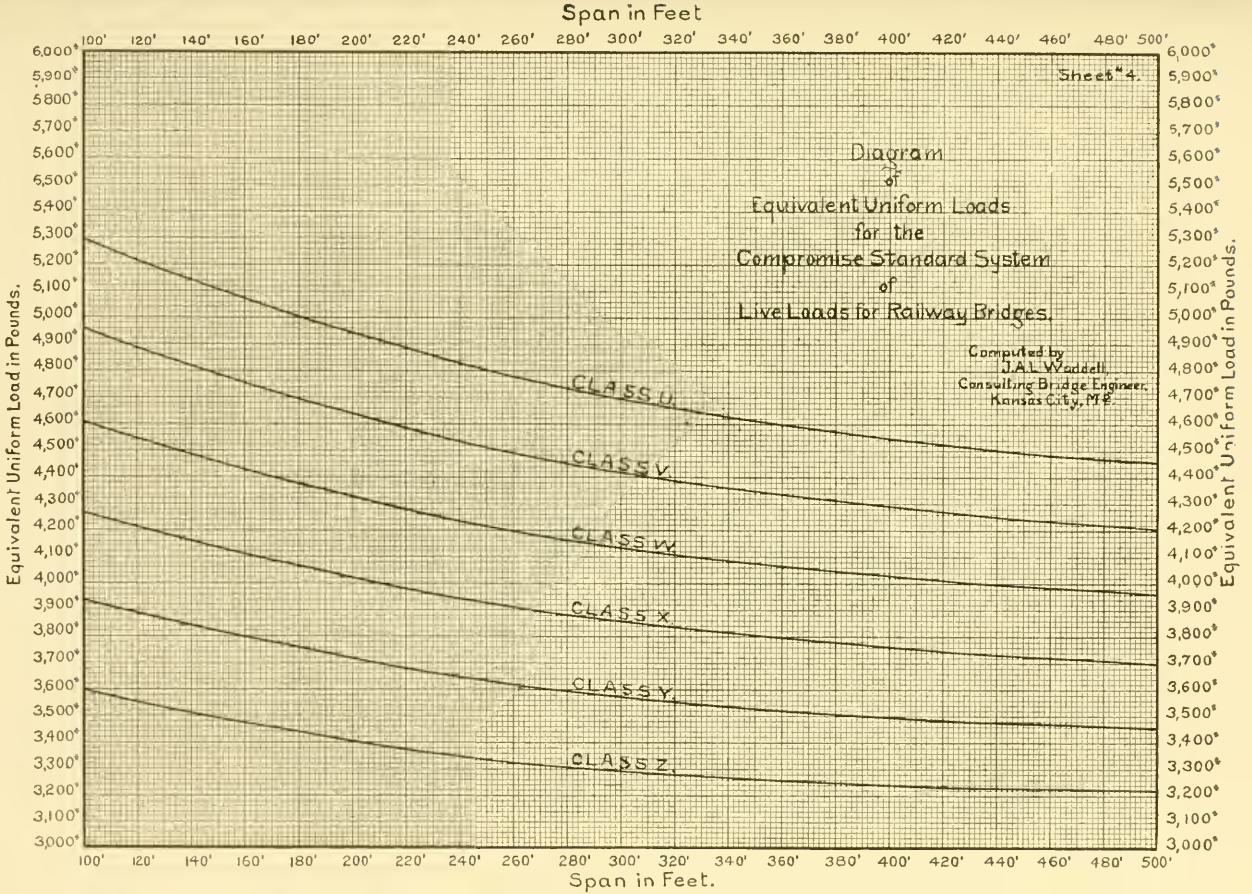


PLATE XXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 632.
WADDELL ON TRAIN LOADINGS FOR BRIDGES.

In order to make the accompanying set of plates complete, I have inserted Plate XXVI, which gives at a glance the end shear from live load on any plate-girder span up to 100 ft. in length for all classes of the "Compromise Standard System."

Just here I desire to call attention again to the short method of finding live load stresses which I first proposed on page 265, Vol. XXVI, of the *Transactions*, and which I have elaborated in the lately published pamphlet before referred to, as follows:

"Where trusses have unequal panels or chords not parallel, the first step to take is the finding of all the dead load stresses by the graphical method, starting from one end of the span and working towards the middle, where the last stress is checked by the method of moments, and the correctness of the entire graphical work is thereby proven.

"The next step is to find from Plate IV the equivalent live load per lineal foot for the span, and therefrom the value of the panel truss live load L. Next set a slide rule for the ratio of dead load per lineal foot, and the equivalent live load per lineal foot for the span, and, by referring to the dead load stresses already found, read off from the rule all of the live load stresses in chords and inclined end posts.

"Next assume that there is an upward reaction at one end of the span equal to 1 000, 10 000 or 100 000 lbs. (according to the size of the bridge), caused by a load placed at the first panel point from the other end of the span; then find graphically the stress in each web member from end to end of span, caused by this assumed upward reaction. Then calculate the value of the live load reaction for the maximum stress in each web member by means of the slide rule and the following formula and table in which n is the number of panels in the span; n^1 is the number of the panel point at the head of the train, counting from the loaded end of the span, and C is the coefficient of $\frac{L}{n}$.

"Live load reaction for head of train at $n^1 = C \times \frac{L}{n}$.

n^1	C	n^1	C	n^1	C	n^1	C
1	1	7	28	13	91	19	190
2	3	8	36	14	105	20	210
3	6	9	45	15	120	21	231
4	10	10	55	16	136	22	253
5	15	11	66	17	153	23	276
6	21	12	78	18	171	24	300

"Then, still using the slide rule, find the greatest live load stress in each web member by the following equation:

$$\text{Stress required} = \text{stress from assumed reaction} \times \frac{\text{actual reaction}}{\text{assumed reaction.}}$$

"Where the panels are divided as in the Pettit truss, and where inclined subposts are employed, the tensile stress in the upper half of each main diagonal thus foundwill have to be corrected by subtracting therefrom a stress equal to $\frac{L}{2}$ sec. A, where A is the inclination

of the diagonal to the vertical. But when inclined subties are used instead of inclined subposts, the correction just referred to will apply only to the compressive stresses in the lower halves of the main diagonals. The reason for making this correction, as will be at once evident to any one who is accustomed to finding stresses in Pettit trusses, is that the method above outlined ignores the subdivision of the panels when ascertaining by graphics the stresses caused by the assumed upward reaction."

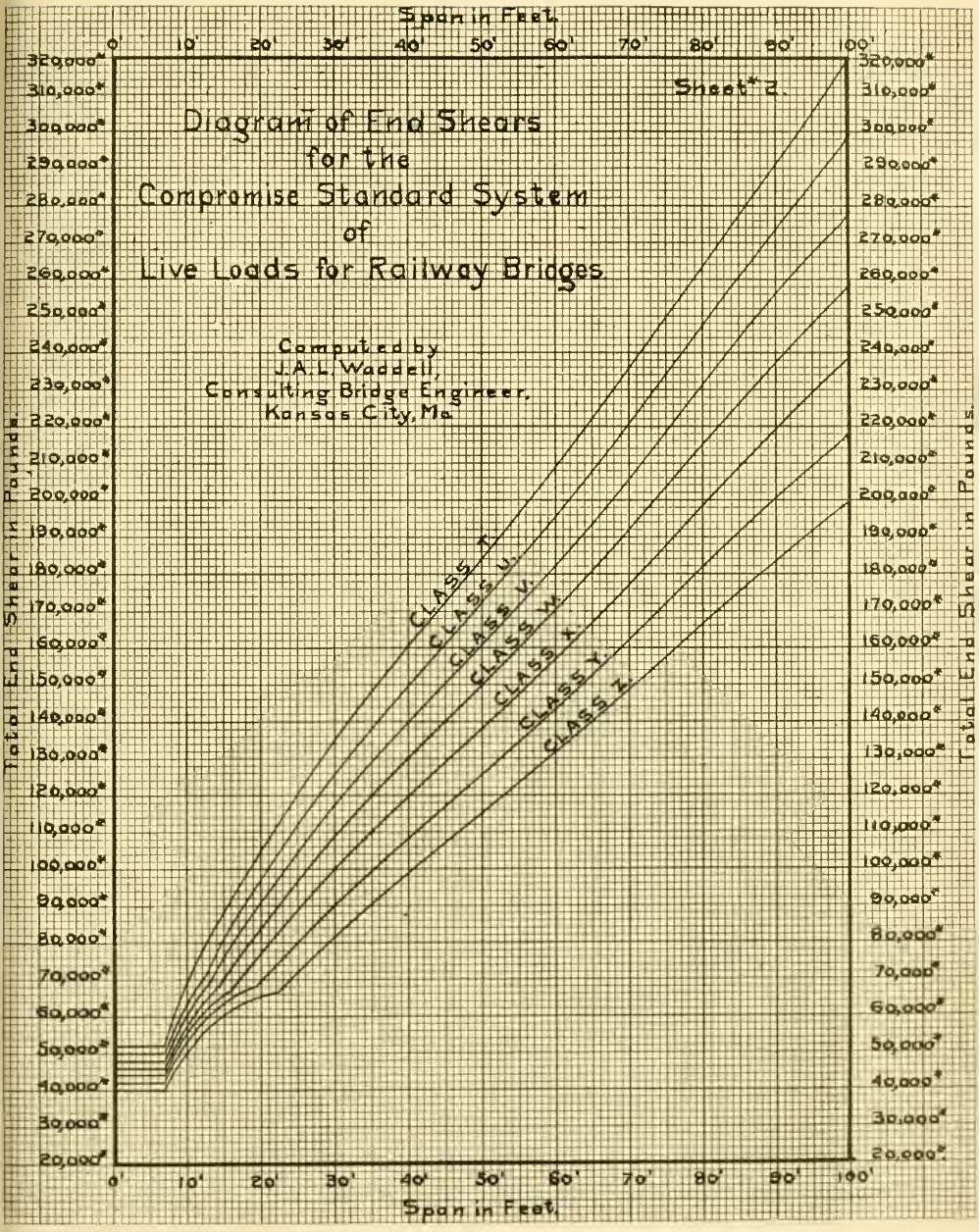
To illustrate the facility with which live load stress calculations are made in this manner by the "Equivalent Uniform Load Method," I would state that this morning I had occasion to compute all the live load stresses in the trusses of a double track, 624-ft. span in 16 panels (after having already found by graphics the dead load stresses), including both the tensile and compressive stresses on every main diagonal and post from end to end of span (caused by the advancing load), and the concentrations on hangers and primary inclined posts; and that it took me to do all this exactly 49 minutes by the watch. To find all of these stresses exactly by the concentrated wheel-load method would take a good computer from one to two days, provided that he ascertain for each truss member the exact position of the wheels to give the true maximum stress.

Again, a few days ago, using the method I am advocating, I figured in one day's work the total weight of metal in a span of 1 100 ft., computing the sectional areas of all truss members. It is true that I had at hand for reference a detailed design for a similar bridge of 1 040 ft. span. Unfortunately, I did not note how much time it took to calculate the live load stresses, but evidently it could not have been very long.

When one considers that stress calculations can be made thus quickly, and always with sufficient accuracy for all practical purposes, by the "Equivalent Uniform Load Method," he surely must come to the conclusion that the old, laborious and hair-splitting concentrated wheel-load method will soon become a thing of the past.

Mr. Cooper's remarks about the limiting weights of engines and cars are sound and very much to the point; for surely the practical limits of said loadings have been nearly, if not quite, reached already. In all of my investigations on the live load question during the last three years, I have failed to find a single actual train load as great as that of "Class U" of the "Compromise Standard System." By the way, I might here state that the extension of that system to "Class T" (which is to be used only for floor systems, primary truss members and plate-girder spans) was made to meet the theoretical but excessive requirements of one or two important lines of road, the bridge engineers of which appear to be endeavoring to discount the future.

Should any one desire to look up the discussion of this live load question since my paper on "Some Disputed Points," was issued,


Span in Feet.
Sheet № 2.
Diagram of End Shears
for the
Compromise Standard System
of
Live Loads for Railway Bridges.
Computed by
J. A. L. Waddell,
Consulting Bridge Engineer,
Kansas City, Mo.
CLASS T.
CLASS U.
CLASS V.
CLASS W.
CLASS X.
CLASS Y.
CLASS Z.
Total End Shear in Pounds.
Total End Shear in Pounds.
Span in Feet.

I would refer him to the back files of the *Railroad Gazette* and the *Engineering Record.*

Mr. Cooper (by letter).—The writer is aware that some members of our profession do not believe in the use of train diagrams in bridge specifications; he not only believes it desirable to continue their use, but also thinks the general practice to be still in favor of that method of specification.

The purpose of the paper was to suggest a series of diagrams based upon the idea of regular gradations from the lightest to the heaviest engines; that the same could be discussed, to see if those who do believe in train loads could agree to some simple series, and thus do away with the many variations now in use.

Mr. Churchill's discussion is in favor of abolishing train loads, and however good his suggestion may be, it does not apply to the paper. The writer fully agrees with him in reference to the desirability of limiting, as far as we can, the standard train loads between 3 200 and 4 000 lbs. But it is beyond the power of any one to compel the use of any particular loading over all roads. Each of us may be able to exert the influence of our opinions over a limited range. It is necessary, therefore, if we desire to treat the matter generally, to make the series sufficiently elastic to cover a wider range than the above.

Mr. Fulton's suggestion to adopt one freight load of 4 000 lbs. is likewise impracticable.

The suggestions of Messrs. Blakeley and Bland as to increasing the relative weights of the tenders are pertinent, But the recommendation to also increase the load on the leading truck does not appear so desirable from the point of view of a typical train load, though it may be nearer actual conditions.

To express the car loads by definite spacings of the axle loads, as recommended by Mr. Bland, would make no difference in the final results, but might be confusing to those buyers of bridges who used cars of a different spacing.

The writer is glad to have the endorsement of such an able authority as Mr. Barnes upon his views in regard to the maximum loads of the future.

The writer regrets that he was unaware that the matter of standard live loads had been finally closed by Mr. Waddell. The writer's ignorance is perhaps due to the fact that he never knew scientific subjects could be settled by ballot. If the profession generally had known this important fact, how much worry and brain substance could have been saved.

It has, however, perhaps mistakenly, been thought that under some circumstances the opinion of one individual on a particular subject might outweigh the opinions of hundreds, and even millions, of other people.

Some years ago, while hearing evidence in a dispute between two large railroad corporations, a Mr. Blank, Chief Engineer of Blank Railroad, testified that he had been an engineer for 35 years, etc., and gave very strong evidence on one side. Upon cross-examination, it developed that the Blank Railroad was three miles long and only partially located; that he had never been connected with any other railroad; and that he had heretofore confined his talents to land surveying.

As Mr. Blank's name may still be among the list of chief engineers in North America, would his ballot on bridge subjects equal those of engineers who have made these subjects their life work?

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications

693.

(Vol. XXXI.—February, 1894.)

PROTECTING PILES AGAINST THE "TEREDO NAVALIS" ON THE LOUISVILLE AND NASHVILLE RAILROAD COMPANY'S LINES.

By R. MONTFORT, M. Am. Soc. C. E.

READ NOVEMBER 1ST, 1893.

WITH DISCUSSION.

Protection by Concrete.—In the year 1880 the Louisville and Nashville Railroad Company acquired by lease the New Orleans and Mobile Railroad, running from New Orleans to Mobile, now known as the New Orleans Division of the Louisville and Nashville Railroad. With this lease it also acquired large creosoting works, located at West Pascagoula River. These works were then, and have since been, operated by Messrs. Putnam* and Tobias.

The New Orleans Division crosses a number of bays, bayous and rivers, all of which are spanned by means of yellow pine creosoted pile trestles, and iron bridges supported by creosoted pile piers.

The total number of linear feet of trestle is 21 407, and of iron bridging 6 459. In considering the effect of the teredo on creosoted

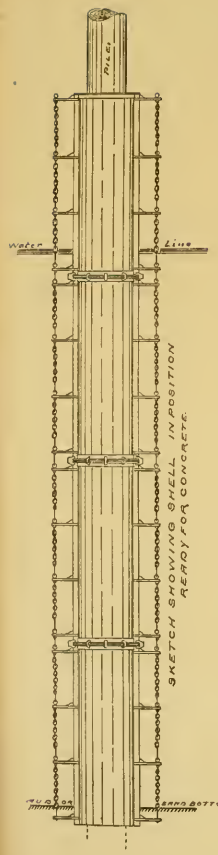
*J. W. Putnam, Assoc. Am. Soc. C. E. (recently deceased).

yellow pine timber, and the further means adopted to protect it against their ravages, the most noteworthy structures are Bay St. Louis and Biloxi Bay trestles. The bridge over the former is 10 241 ft. long, and consists of 10 050 ft. 2 ins. of creosoted pile trestle, with a 190-ft. 10-in. iron draw span ; that over the latter is 6 040 ft. 10 ins. long, and consists of 5 850 ft. of creosoted pile trestle and a 190-ft. 10-in. iron draw span. These trestles were built by the New Orleans and Mobile Railroad Company in the years 1878–79, and replaced structures destroyed by the teredo, which abound in the waters of the Mexican Gulf and its estuaries, and are extraordinarily destructive on account of the long, warm season during which they work. It is no uncommon occurrence for the teredo to completely honeycomb an untreated yellow pine pile of from 12 to 15 ins. diameter in less than six months, so as to render it unsafe for structural purposes. In 1871 a serious accident occurred from this cause at Biloxi Bay Trestle, when a freight train went through the bridge, although the piles were only about ten months old ; seven bents went down with the train. An examination showed that the piles were all eaten off close to the bottom of the water. When first built, Bay St. Louis Trestle had hardly been completed when it was found the untreated piles were so badly attacked by the teredo that it was necessary to commence rebuilding at once. In 1872 resort was had to covering the piles with copper before driving, and this gave better results; but the protection was not found to be perfect. It was, therefore, decided in 1876, to construct the creosote works already mentioned, and they were built at a cost of $60 000.

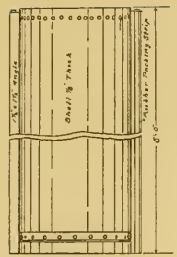
A close watch has always been maintained over the creosoted structures, in order to determine their behavior and economy as compared with similar structures of untreated timber. In addition to periodical inspections of those portions above water, the piles from the surface of the water to the top of the mud or sand, which composes the bottom, have been examined from time to time by divers. It was not, however, until 1886 that it was discovered the teredo had commenced its attacks on the creosoted piles. About the same time an inspection of the creosoted piles in the railroad company's wharves at Pensacola, Fla., which had been built in 1880, disclosed a similar condition of affairs.

In view of the vast amount of creosoted timber that existed in the structures on the New Orleans Division, already referred to. and also at Pensacola, Fla., and on the Pensacola and Atlantic Railroad,

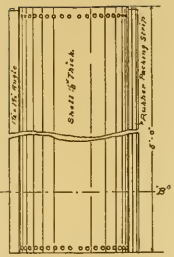
PLATE XXVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 633.
MONTFORT ON PILE PROTECTION.



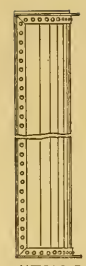
SKETCH SHOWING SHELL IN POSITION READY FOR CONCRETE.



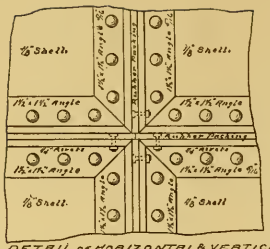
INSIDE ELEVATION
HALF BOTTOM SECTION



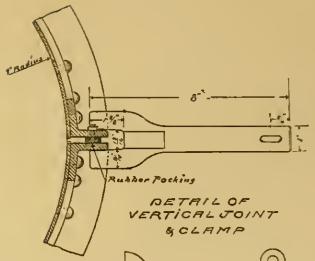
INSIDE ELEVATION
HALF INTERMEDIATE SECTION.



OUTSIDE ELEVATION.
INTERMEDIATE SECTION.



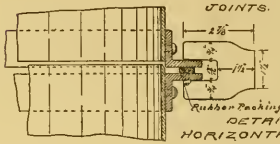
DETAIL OF HORIZONTAL & VERTICAL JOINTS.



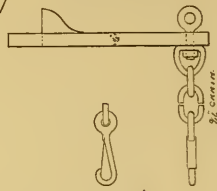
DETAIL OF VERTICAL JOINT & CLAMP

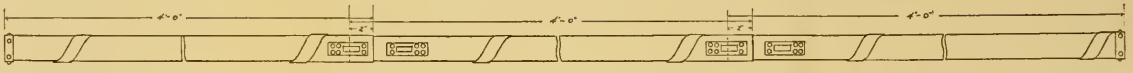
TOP VIEW OF BOTTOM SECTION

SECTION 'A' 'B'

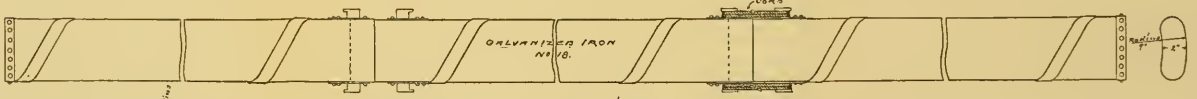


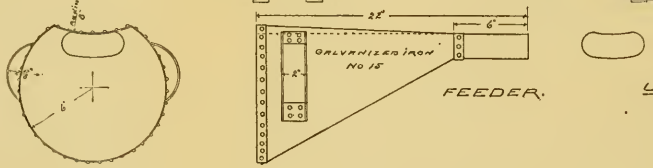
DETAIL OF HORIZONTAL JOINT & CLAMP.





DUMPING PIPES.





FEEDER.

WROUGHT IRON SHELLS
USED IN PROTECTING PILES WITH CONCRETE
ON THE L.&N.R.R.

which was built in 1882–83, and is owned by the Louisville and Nashville Railroad Company, the question of determining, if possible, on some further means of protecting the creosoted piles against the teredo was of vital importance. In connection with the late F. W. Vaughan, M. Am. Soc. C. E., Consulting Engineer, the writer was instructed to investigate, experiment and report, with recommendations of what should be done. As a result, it was decided to adopt a thin coat of cement mortar or concrete, applied to the outside of the piles from the surface of the mud or sand at the bottom to the surface of high water at the top. In order to accomplish this, the work was conducted in the following manner : A shell of wrought iron, made in circular form, composed of several sections, each in two segments, so arranged as to be easily separated (see Plate XXVII), was placed around the pile ; the shell was clamped together above the water and lowered, one section after another, until it completely surrounded the pile from the surface of the water to a distance of from 6 ins. to 2 ft. below the bottom, varying with its hardness and the difficulty of forcing the shell down. A diver placed a pudding of "gumbo clay," inclosed in sacking, between the shell and the pile at the bottom before the shell was forced down, thus making the space between the pile and the shell almost water-tight, where the depth of water did not exceed 12 ft. The water was then pumped out and the mortar, or concrete, poured in. Where the depth of water exceeded 12 ft., it was found impracticable to keep the water out, and the attempt to do so was given up. In such case the mortar, or concrete, was passed down to the bottom of the shell through a galvanized iron pipe of special shape (see Plate XXVI), so as to give as large an opening as possible without taking up too much space between the pile and the shell. A funnel or feeder was used at the top of the pipe, to render it easy to fill. The pipe was gradually raised as the concrete filled up the space between the pile and shell ; but the lower end of the pipe was kept constantly in the concrete, so as to prevent the concrete from falling through the water, which would separate its constituents, and, of course, ruin its adhesive power. The shell was allowed to remain three or four days, until the concrete had set, when the clamps were pulled off, and it was removed and placed on the next pile to be treated in a similar manner. Shells made of wood, held together with cast-iron bands, were also used extensively on account of their relative cheapness as compared with

showing section covered with Sea Weed

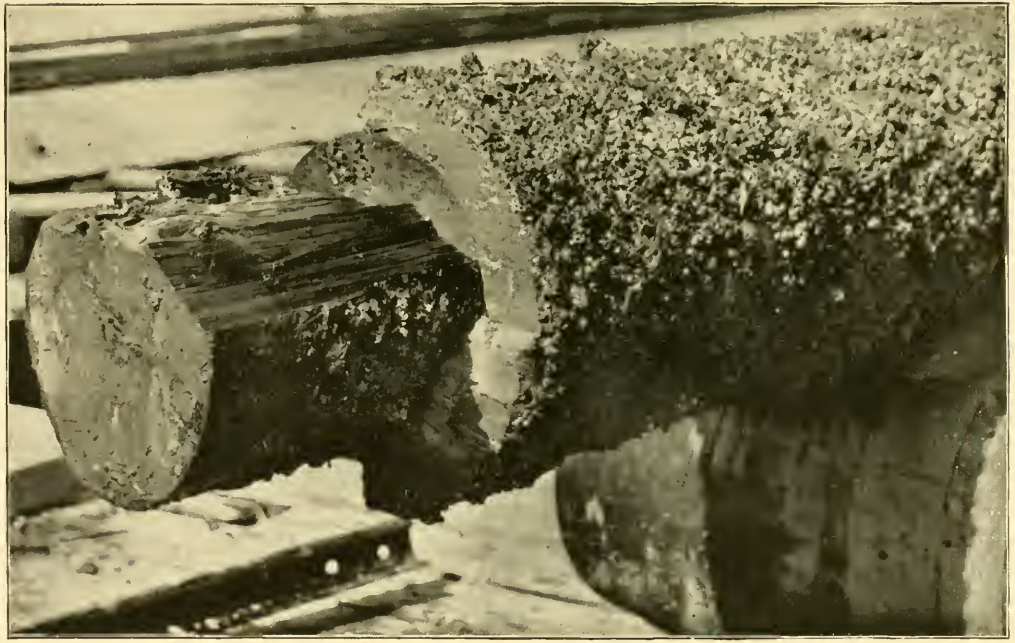
the wrought-iron shell. In this way 2 638 piles were protected in the wharves at Pensacola, 840 in Escambia Bay Trestle, 413 in Biloxi Bay Trestle, and 216 in Bay St. Louis Trestle, making a total of 4 107 piles. It was only applied to piles that were found to be seriously attacked by the teredo, and that, if not protected, would soon be utterly destroyed. One important advantage which this means of protection possesses, is that it can be accomplished without in any way disturbing the piles or the superstructure which rests on them, and this was especially important in the case of Pensacola wharves on which large storage sheds had been built. If the piles had to be replaced by new piles, the floors and roofs of these buildings would have greatly interfered.

It is now seven years since the work of protection, as described, was begun, and although some little trouble has been experienced from logs and rafts striking the protected piles in rough weather and abrading or cracking the concrete on a few of them, the expense of repairs has been small, and all of the piles which have been protected are still in use, and likely to continue so for a number of years; whereas, if they had not been protected, it would have long since been necessary to replace them by new piles. Plates XXVIII and XXIX show piles broken off below the surface of the mud, and taken up at Pensacola wharves three years after protection. They were found to be in good condition. Plate XXVIII shows a pile before the concrete was broken; Plate XXIX, after it was broken and the pile sawed in two. Not only did the concrete completely cover the piles, and was itself covered with oysters, barnacles, etc., but the cement had found its way into the teredo holes, filling them even to the heart of the pile, and forming within it perfect casts of the teredo.

To remove the concrete required several heavy blows with an axe.

The piles were finally split up into small pieces, without finding a single living teredo.

The concrete was composed of sand, gravel and "Alsen's" Portland cement, in the proportions of 1 of cement, 2 of sand and 3 of gravel; sufficient water was used to render it thin enough to readily pass through the pipe. When the concrete was broken off, many of the fractures passed through the centers of the flinty gravel, showing the great strength with which the cement adhered.



Pensacola Wharf.

Section of pile showing Old Teredo Holes, Concrete protection covered with Barnacles, Oyster Shells and Sea Weed.

PLATE XXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 693.
MONTFORT ON PILE PROTECTION.

The work of protection was first started at Pensacola, under the immediate charge of Superintendent E. O. Saltmarsh, to whom, as well as W. H. Courtenay, M. Am. Soc. C. E., Principal Assistant Engineer, a large part of the credit for the success of the undertaking belongs.

The cost of protecting piles in this way was found to vary from 80 cents per linear foot of concrete, measured on the pile, to $1 50 per linear foot, depending upon the length of the protection applied, the location of the pile, and the conditions of the weather ; the average was about $1 25 per linear foot. But as the number of feet that required protection—viz., from high water-mark to the surface of the bottom—was small compared with the total length of piles, the expense was much less than what it would have cost to have replaced the old piles with new creosoted piles.

Protection of Vitrified Clay Pipe.—Last year, 1892, a careful surface inspection and inspection by diver showed the following condition of the piles at Bay St. Louis and Biloxi Bay trestles :

	Bay St. Louis. Number piles.	Biloxi Bay. Number piles.	Total Number piles.
Not attacked	618	334	952
Few small holes	225	52	277
Full of small holes	567	273	840
Large holes	997	576	1 573
Piles already protected by concrete	216	413	629
*Decayed	3	....	3
Totals ...	2 626	1 648	4 274

*Replaced by new piles. Probably not sound when treated at the time trestle was built.

From the above it will be seen that a very large number of piles in both of the trestles needed immediate protection, and, as the caps and stringers were too light for existing rolling loads, it was decided to strengthen them at the same time that the piles were being protected, and to make a complete job by protecting all of the piles not already protected by concrete.

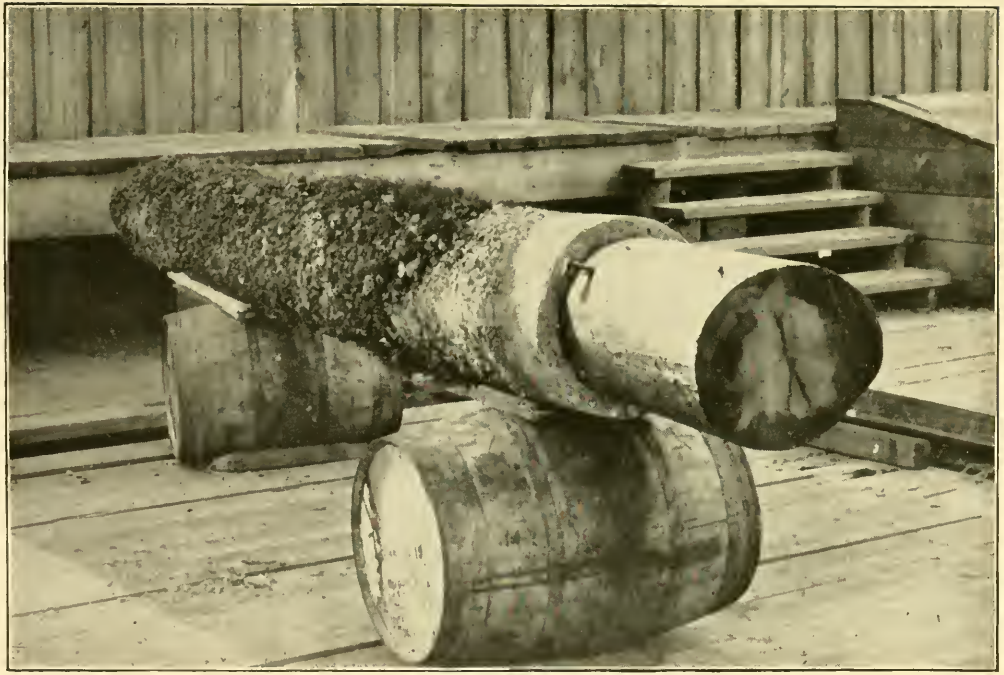
These trestles have been in service for 14 years, yet the timber was found to be in a good state of preservation from decay, with the exception of a few ties, which were replaced by new cypress ties (see Plate XXX). It is expected that the life of the new cypress will be as

long as the remaining life of the old creosoted pine above the water-level, and that when renewal above that level is required—10 or 12 years hence—a frame trestle can be placed on top of the protected piles, as indicated on the drawing.

It will be noticed, by reference to the drawing, that, instead of using concrete protection on these two trestles, vitrified clay pipe was substituted for it. The reason for this was two-fold—first, on account of economy ; secondly, because it was believed that the pipe would be a more certain protection to the pile immediately above the surface of the mud, it having been found, in a few cases, that the mud had become mixed with the bottom of the concrete, and, of course, killed the cohesive power of the cement, thus leaving 2 or 3 ins. of the pile above mud line exposed to the attacks of the teredo. In most localities this is not of vital importance, as the teredo usually works near the surface of the water and between high and low tide, his ravages becoming less as the bottom is approached. But at Biloxi the case is reversed, due to fresh water flowing on top of the more dense salt water, hence the teredo thrives better near the bottom, and it is quite common to find piles there entirely eaten off at the surface of the mud, while showing no indication of having been attacked at the water-surface or between high and low tide. To this may be attributed the accident of 1871, already referred to. The same is true to a less extent at Bay St. Louis.

In order to allow the sections of pipe to be readily passed over the tops of the piles, it was desirable to cut the piles off at a sufficient distance below the stringers. This necessitated the use of a bolster, or some other contrivance, to support the stringers, and, as it was also necessary to strengthen them, the bolster was made to extend the full panel length and its upper surface keyed to the lower surface of the stringers, with cast-iron keys, so as to make it act as far as possible in conjunction with the stringers. This arrangement also permits the easy removal of either of the sticks composing the stringer, or of the bolster, independent of each other, in case any one of them should decay before the others.

A great deal of trouble had been experienced at both of these trestles, on account of fires caused by sparks dropped from the engines of passing trains during droughts, and it was therefore determined to box in the floor and cover it with gravel.



Pensacola Wharf.

End view of pile protected with concrete,
Pile removed three years after protection.

PLATE XXIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 693.
MONTFORT ON PILE PROTECTION.

The sections of pipe were fastened together with a composition of pitch and sand, applied hot. As each joint was lowered below the surface of the water, the composition was cooled by the water and became hard and held the two sections of pipe firmly together. A third section was then added, and so on, until the pipe reached from a solid bearing, 1 or 2 ft. below the mud bottom, to high water-mark. Jacks were used to force it down. The space between the pipe and pile was then filled with sand, which was allowed to settle. Finally, after several months, the pipes were again refilled with sand, to make up for settlement and that which waves had washed out, and were then sealed on top with the same composition used for cementing the joints of the pipe.

The cost of protecting piles by means of pipe was found to be 66 cents per foot of pipe where 16-in. pipe was used, and 71 cents per foot where 18-in. pipe was required. These figures represent everything connected with the work, including freight on pipe from St. Louis, Mo., to bridge site—quite a large item. The length of pipe used on each pile varied from 15 ft. at the centers of the bays to 2 ft. 6 ins. near the shores. As the piles were from 50 to 60 ft. long, the cost of protection was much less than would have been the cost of renewal with creosoted piles.

A test pile, known to be infested with the teredo in a live and healthy state, was treated in this way by having a pipe placed around it, without filling with sand. At the end of 24 hours it was found that the teredos were all dead and turning black. At the end of an additional 48 hours the pile was again examined, and the bodies of the teredos had almost disappeared. At the end of an additional week it was again pulled up, and split into pieces, and nothing could be found of the teredos, except the holes that they had left ; all parts of their bodies had disappeared. With the space between the pipe and pile filled with sand, we can therefore feel assured of the utter destruction of the teredo and the permanent protection of the pile from their further ravages. The result of the experiment might fairly be inferred from the fact that the teredo never attacks a pile below the surface of the mud, or sand, into which it has been driven.

In cutting off the 4 274 piles at both trestles, but three were found to be decayed, which, considering their age (14 years), is quite a good record for yellow pine timber treated with dead oil creosote. From

all indications they will be good for at least 10 years' further service. Untreated yellow pine piles would not last in this climate more than an average of seven years, assuming that the teredo did not eat them up, as it would in far less time.

* * * * * * * * *

Since the foregoing was written, the Gulf coast was visited by the most disastrous hurricane known in its history, which caused the death of some 2 000 people and inestimable damage to all kinds of property. The Louisville and Nashville Railroad Company was by no means exempt. An idea of the force of the hurricane can be gained by noting the damage all along the line from Flomaton, Ala., to New Orleans, La., a distance of 202 miles. For that distance immense numbers of trees, many measuring over 2 ft. in diameter at the butt, were twisted off and blown down across the track. Telegraph poles and telegraph wires were also blown down.

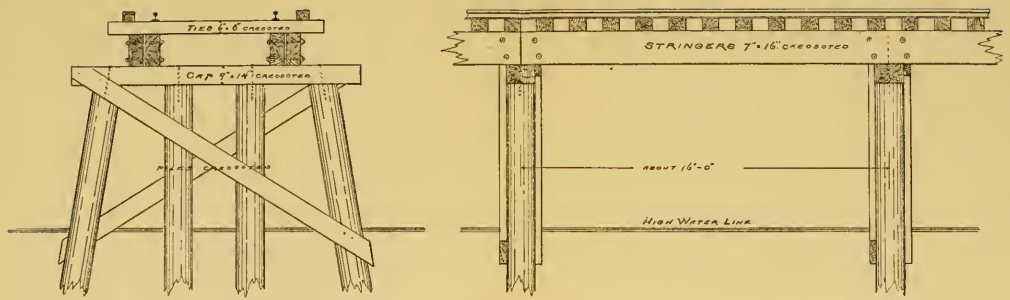
From Bayou Sara Bridge, nine miles north of Mobile, to Mobile, the track was covered with heavy drift. Trees of large dimensions and saw-mill logs, 60 to 70 ft. long and 30 ins. square at the butt, were piled one on top of the other to a depth of from 4 to 8 ft. The track was also washed off the road in some places. Chickasawbogue Draw Bridge was moved bodily 6 ins. out of line. In Mobile the water was 4 ft. deep in the Union Depot, and the same depth on the floor of the Cotton Exchange. Large flat boats were stranded in the middle of the streets, and many dead bodies were floating in the river. Fortunately there was only one train between Bayou Sara Bridge and Mobile ; it was side-tracked at Magazine Point and was protected from the storm. None of the crew lost their lives, but a section-man in the same locality was not so fortunate; he was carried off the roadbed and drowned.

The New Orleans and Mobile Division was a much heavier sufferer. From East Pascagoula Bridge, at Scranton, to West Pascagoula Bridge, at West Pascagoula, a distance of three miles, the entire track—rails, fastenings and ties—was carried off the roadbed and into the marsh, two to three miles away. The roadbed at this point is about 10 ft. above ordinary high tide, and no serious trouble had ever been experienced with it from the time the road was built until October 2d, 1893. Three large schooners were landed on the embankment, one in the center of the

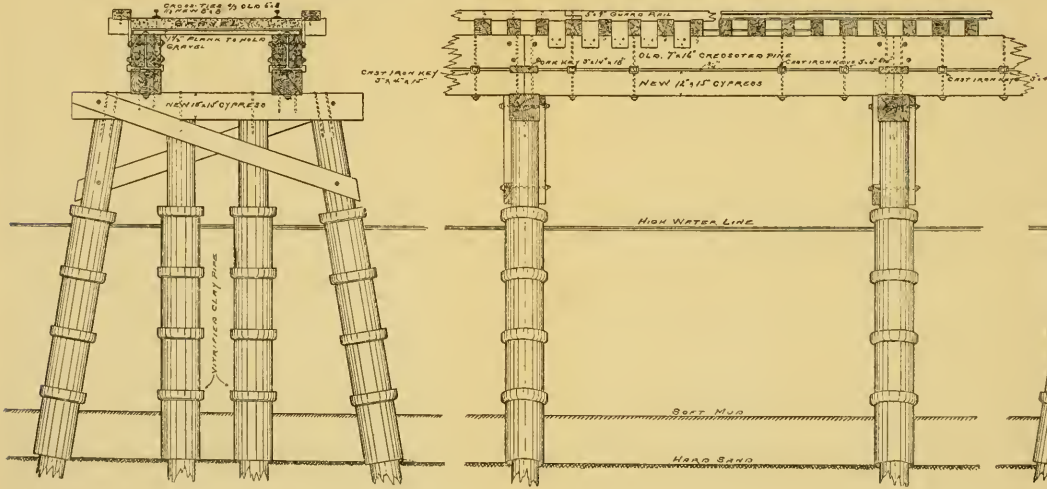
PLATE XXX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 693.
MONTFORT ON PILE PROTECTION.

METHOD OF STRENGTHENING AND PROTECTING TRESTLES ON THE N. O. & M. DIVISION OF THE L. & N. R. R.

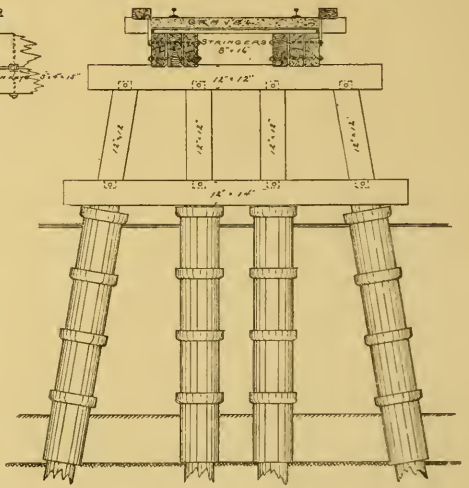
OLD TRESTLE



TRESTLE WITH DECK STRENGTHENED & PILES PROTECTED.



PROPOSED METHOD OF PLACING FRAMED BENT ON OLD PILES WHEN SUPERSTRUCTURE NEEDS RENEWAL.



roadbed, the other two about half way up the bank, which was badly washed by the waves.

At East Pascagoula Bridge one end brace of a 77-ft. iron span was bent so badly as to render the structure unsafe. The floor of the entire bridge, 700 ft. long, was thrown out of line; wrought-iron tension members of all of the spans were bent and kinked, but not to a sufficient extent to render them dangerous or to make repairs difficult or expensive. At West Pascagoula one 97-ft. span was completely wrecked; the iron phœnix column posts were broken to pieces, and the superstructure, with exception of the floor, had dropped into the river. The floor was hanging to the rail, which did not break, but sagged down about 6 ft. The floors of the other spans of West Pascagoula, 1 350 ft. in length, were in bad shape—knocked out of line and up against the trusses by the force of the waves and drift.

At both ends of Biloxi Trestle the embankment was washed away, as was also the floor of the trestle, with the exception of 975 ft. at the north end, which was intact; the remainder, about 4 875 ft., including the caps, stringers, ties and rail, was gone. All that part south of the draw was afterwards found from one to three miles up the bay; the material north of the draw was found several miles down the bay.

The hurricane first blew the water into the bay, and afterwards wheeled around and blew it out again.

The draw-tender, Mr. Vanderpool, and his assistant, were in their house at the draw at the time the storm came up and remained there until about 6 A. M., when they became frightened and attempted to escape by means of the trestle, which was not damaged up to that time. The assistant draw-tender had only proceeded about 20 ft. when a wave struck him and carried him away. His body has not yet been recovered. Vanderpool, the draw-tender, then abandoned the attempt to escape and climbed up into the trusses of the draw span and tied himself on with a rope. While in this position he had full opportunity of observing the effects of the storm, which did not reach its maximum for some time after he and his assistant had attempted to go on shore. He states that the waves rose many feet above the top of the trestle and the floor of the draw, and that boats were carried clear over the trestle on these waves. The bay was full of drift, the accumulation of many years in the marshes. This drift consisted of immense trees, which struck the trestle and bridge with enormous force,

battering the piles and caps. and finally, by continued pounding, as each wave struck them, knocked the caps loose from the piles, when the floor of the trestle was carried away. This allowed the drift to escape, and it finally went to sea. The condition of the trestle, after the storm, fully confirms Mr. Vanderpool's statements. The tops of the piles, as well as the caps, showed the heavy blows to which they were subjected by the drift striking them. Seventy-seven piles were entirely broken off so they cannot be used again; many of them were carried away and are nowhere visible. Other piles, where the drift bolts did not pull out, were burst in two.

The caps were secured to the piles in the usual way, by means of drift bolts. An additional drift bolt was, however, used in the two outside piles, making a total of six drift bolts, ¾ in. in diameter and 24 ins. long, to each bent.

When the floors of Bay St. Louis and Biloxi trestles were strengthened, the grade of the track was raised in low places, in order to remove depressions which previously existed in the track. No change was made in the method of securing the caps to the piles, as prior to the hurricane of October 2d no trouble had ever been experienced with caps similarly secured. Had extraordinary means of securing the caps to the piles been adopted, such as wrought-iron straps fastened by cross-bolts through the piles, it is very doubtful whether the trestle would not have been in a worse plight than it was; in other words, it is more than probable a very much larger number of piles would have been broken off and carried away, had not the detachment of the floor liberated the drift which was pressing against the piles and which broke 77 of them. In this connection it is worthy of note that a large proportion of the broken piles that are visible have the caps still attached to them. Had the piling at Biloxi and Bay St. Louis trestles been lost, it would have taken months before the road would be open for through traffic.

The floor of the draw-tenders' house was destroyed and the whole building badly damaged. This building is supported on piles and is level with the draw. The floor of the draw was also damaged, and a number of the floor-beams carried away, while others were detached from their places. These floor-beams were large sticks of timber 8 x 16 ins. by 17 ft. long, resting on the wrought-iron bottom chord of the truss, and were secured by two bolts 1¼ ins. diameter. The force

with which they were struck must therefore have been tremendous. The lateral bracing between the lower chords of the draw was also broken in several places, and the center cast pedestal, on which the weight of the draw rests, was cracked.

The tops of the pile piers, which supported the ends of the draw, were carried away.

At Bay St. Louis Bridge the embankment at the north end was washed away. The floor of the trestle for 7 170 ft. was carried off in a similar manner to Biloxi, leaving 2 880 ft. at the south end of the trestle intact. A schooner had crushed through the trestle, carrying with it about 48 ft. of the floor, as well as the piles supporting it. The drift at Bay St. Louis was tremendous. One saw mill alone is reported to have lost 10 000 logs, all of which went through the trestle and out to sea. In addition to saw-mill logs, there were acres of drift from the marshes, composed of immense trees such as those already described, and one of these, over 60 ft. long, still remains tangled up in the piles. All of the bathing houses, both at Biloxi and Bay St. Louis, were carried away early in the storm, as well as the timber piers leading to them; they helped to add to the accumulation of drift against the trestle.

The drawbridge at Bay St. Louis was found in a similar condition to Biloxi drawbridge, already described; many of the floor-beams gone and others knocked loose and up against the trusses.

The draw-tender and his assistant at Bay St. Louis were, however, more fortunate than those at Biloxi. They became alarmed earlier and succeeded in escaping by crawling and holding on to the rails when the waves broke over them.

The floor of Bay St. Louis Trestle was carried out to sea. A large part of it was stranded on Ship Island, some 15 or 16 miles distant, and this it is expected to recover.

Forty-five piles in Bay St. Louis Trestle were gone or completely broken, making a total for both trestles of 122 piles destroyed out of a total number of piles in both trestles of 4 274: that is, the number destroyed was a little less than 3% of the total number of piles in the trestles. The remainder of the piles can be used again, although many of them are badly battered and split.

The pile protections at both trestles suffered to some extent from being struck with the drift, but the damage is much less than one

would expect. The amount of pipe which will have to be provided to make repairs is only 5% of the total amount of pipe originally used in protecting the piles, and this includes protection of the new piles used in replacing those that were entirely destroyed.

The repairs that have been made to the trestles since the hurricane are of a temporary character, sufficient to permit of the resumption of traffic. When permanent floors are again placed on them, the grade of track will be further raised in the lowest parts about 8 ins., which can be done at slight expense. The draws will be raised a similar amount. The caps will be anchored to the piles with wrought-iron straps and bolts, in addition to the drift bolts, and will be cut off flush with the face of the outside battered piles, so that it will be impossible for drift to catch under the caps.

Extra heavy bolts will be used for securing and binding the stringers, caps and ties together, so as to better provide for storms hereafter; but even with all these precautions, if a hurricane such as that of October 2d should again occur, there is no assurance that any trestle which it is practicable to build will stand the strain.

South of Bay St. Louis, eight miles of track were carried entirely off the roadbed, together with the sand ballast. In some places the marsh had floated over the roadbed, and when the water went down left a deposit of from 3 to 4 ft. in depth.

At Pearl River the iron bridge was in bad condition; trusses were about 2½ ft. out of line, and the floor more than that. The draw-tender stated that at one time during the storm a pile pier, which supports two of the spans, was 10 ft. out of line, but after the wind turned and the drift was carried away, it came back to within 2½ ft. of its original position. Two of the piles in this pier, each about 18 ins. in diameter, were entirely broken off; the iron top had been lifted off the piles and knocked up stream. Many of the truss-bars were bent, and the whole floor was out of place.

The loss of life in and around Pearl River was very great.

The work of repairs was commenced immediately after the storm, and the road was open for through traffic on October 17th.

Diligent inquiry among the older citizens and fishermen residing in the vicinities of Bay St. Louis and Biloxi, as to when high water or a hurricane similar to that of October 2d had previously occurred, developed the fact that in 1860, and also in 1852, the water had been as

high, if not higher; but on both of these occasions the rise was gradual, occupying from three to four days; whereas, in the recent hurricane the water rose in a few hours. No one recollected so severe a hurricane as that of October 2d, 1893, and it is certain none ever occurred that caused the same amount of damage and loss of life.

DISCUSSION.

GEORGE H. THOMSON, M. Am. Soc. C. E.—This paper is interesting to me for the reason that while Resident Engineer at Bay St. Louis in 1869 the building of this bridge was begun. The Mobile line running from New Orleans to Mobile, 140½ miles, was located by the late Henry Van Vleck, a pupil of the late John B. Jervis, M. Am. Soc. C. E., and is noted for its long tangents, its easy gradients, its 30 miles of marshes and 6 miles of bridges mostly subject to the sea worm. The temporary character of the bridges was incidental to hasty construction. In 1869 a pile in salt water with worms was thought to be efficient for three years.

Taking the Bay St. Louis bridge for an example, the water in the bay was fresh up to June 1st, owing to the high water in Bayou Jourdan due to a rainy spring; the worms were not active that year. In the year 1870 the worms were at work, and the specimen of this piece of riddled pile is the work of about, say, 90 days of teredo. None of the holes much exceed $\frac{3}{16}$ in. diameter, yet the piece is fairly honey-combed, and is now of no structural value.

The first Bay St. Louis bridge was a trestle of 16 ft. bents, resting on five piles per bent; the rails for two-thirds of its length were about 7½ ft. above ordinary water.

When the worms (in 1870) began to attack the bridge, the railroad company began the protection of piles about in this way: The piles, for the water length, were worked down to about 12 ins. diameter; then covered with a course of felt; then yellow metal covering was applied, the seams being closely nailed. The protection of Biloxi Bay piling was done with zinc instead of yellow metal. The zinc covering in about one year began to fail by corrosion; the yellow metal was more successful. Other attempts at protection were tested. Piles were charred for the water length of pile and then while hot were soaked with dead oil; then, when cold, black varnish was applied to "hold" the charring firm under the jar of pile hammer. I heard some years after that this work was a success so far as the protection from the

worm, but the piles' heart rotted above the water-line in about six years.

I have seen one worm only that had a diameter of $\frac{5}{8}$ in.

A boat infested with worms will sometimes get rid of them by a two weeks' sojourn in fresh water; this does not always result in the destruction of the worm. Old residents say that worms born in the vicinity of bayous are killed by "tying up" or immersion in fresh water. Very salty water and a hot season find energetic worms, and three months is long enough to fairly riddle an 18-in. pile in some situations.

James Duane, M. Am. Soc. C. E.—The teredo in the Gulf water seems to accomplish in three months what ours here will in two years; it is a question whether this is due to the temperature of the water or the size of the worm. Probably they vary with the kind of timber they frequent, but in the cases I know of they have been very small, perhaps not over $\frac{1}{16}$ to $\frac{1}{8}$ in. in diameter and $\frac{3}{4}$ to $1\frac{1}{4}$ ins. long; we did not find that the depth of water made much difference. A heavy oak timber box covering the water pipe laid to supply North Brother Island was eaten away in two years; it may have been partially destroyed much earlier than that; the break occurred in 65 to 70 ft. of water.

George Hill, Assoc. M. Am. Soc. C. E.—Speaking of the size of the worm as affected by the warmth of the water, ten years ago I knew of some worms in the south branch of the Shrewsbury that attained a diameter close on $\frac{1}{2}$ in.; they went through a pile 8 ins. in diameter, driven to hold the oyster boxes in place, the water not exceeding 4 ft. in depth in about 18 months.

In connection with the life of the pile, Mr. Thomson speaks of the charring as leading to the rotting of the interior of the pile. Mr. William Kennish, connected with the Carolina Oil and Creosote Company, published something relating to charring under high pressure. He illustrated in *Engineering News* a section of pile that had been treated with creosote and some that had been charred first and then treated with creosote. Those treated with creosote showed patches where the worms had gone into them very seriously, while those that had been first charred and treated under pressure were perfectly sound. The explanation was that the charring removed some of the inner portion of the bark, which interfered with the proper impregnation of the pile. It occurred to me that perhaps that was the explanation of the deterioration of the piles at Bay St. Louis and at Biloxi. As I have always labored under the impression that a properly creosoted pile was indestructible, the discussion has been interesting to me, as raising the question of what caused it, and why they did not act in accordance with our other experience in the matter. If Mr. Thomson has had any experience in that particular line, I would be much interested to know about it.

Mr. Thomson.—In regard to the charring of piles, a man in Texas first suggested the use of charring. He tried them, and, so far as he could see, they were successful. The piles rotted inside, above the water, in six years, as they would anyway.

I tried from 1869 to 1872 all sorts of patents for the protection of timber. The worms seemed to attack the patent timber sooner than the natural, with one exception. In 1866 I was a student of chemistry with Dr. George Hadley, of Buffalo. He told me that he had spent a part of his youth in Bay St. Louis. He sent me a piece of his treated timber in 1869, which, up to January, 1873, had not been touched by worms.

The paper describes a great storm. In 1869 there occurred just such a storm at Bay St. Louis. The sea was at first very calm, but the sky was dark, and a low rumbling noise could be heard as the storm approached the shore. The water raised 7½ ft., the trees were uprooted, all the bathhouses washed away, and the leaves, 500 ft. from the shore, stripped from the trees. Some of the old darkies told me that their recollection was that these storms came after every other epidemic of yellow fever; the yellow fever came every 14 years, the storms every 28 years. After this storm I thought the grade of the bridge should be raised.

I think in the present state of the iron market we could put in permanent bridges and maintain them a little cheaper than is done now.

A. McC. Parker, M. Am. Soc. C. E.—In 1871, the teredo was very plentiful in the waters of this harbor, and the life of piling in the clear water near the pier heads was not over about 10 years. At that time all of the sewers discharged at the bulkheads, and, while the water close in shore was foul, but little of the sewage was carried out near the heads of the piers. The work of extending the sewers in boxes under the piers was begun about 1872, and at this present time nearly all of the sewers on the North River, and a good many on the East River below Fifty-ninth Street, discharge their sewage well out in the stream. It has been found, as the sewage is carried out into the river here and there, so that it pollutes the water near the pier heads, that the teredo is not nearly so destructive, and it seems to be disappearing in the waters of the Hudson. It is still to be found there, but smaller and less ravenous, and fewer in number than a few years ago. Evidence is not wanting to show that this is so, and I may cite the case of the derrick *City of New York*. She was docked in the fall of 1878, and it was found that the yellow pine sheathing on her bottom and sides was almost riddled by the worms.

She has recently been on the dock, and I could find but very few holes in the sheathing after 15 years' exposure to the attack of the worms; and it was not removed, but left on her, being about as good

as new. I have taken occasion to examine a great many piles in the last few years, and I do not find that the worm is dangerously active at the present time, and it would seem as if the pollution of the water by the sewage was effective in stopping the growth of this pest.

Various protective experiments were tried by the Engineer-in-Chief of the Department of Docks, and I am permitted to give the following table showing their efficieney. In May, 1878, certain pieces of wood were put down at the end of Pier 1, N. R., in groups, as follows:

GROUP No. 1. SPRUCE, 4 x 10 INS., 12 INS. LONG.

No. 1. Plain, as cut from plank.
No. 2. Studded with 12 lbs. of carpet tacks.
No. 3. " " 8 lbs. of three-penny nails.
The surface of Nos. 2 and 3 entirely covered with nails.

GROUP No. 4. PART OF WHITE OAK PILE, 10 x 11 INS. DIAMETER, 31 INS. LONG.

No. 10. Plain.
No. 11. Studded with 35 lbs. of carpet tacks, driven about ¼ in. apart.

GROUP No. 5. PART OF PINE PILE, 11½ x 12 INS. DIAMETER, 36 INS. LONG.

No. 14. Plain.
No. 12. Studded with 20 lbs. of bellows nails and 12 lbs. of carpet tacks, driven about ¼ in. apart.

GROUP No. 6. PART OF SPRUCE PILE, 13½ x 14 INS. DIAMETER, 36 INS. LONG.

No. 15. Plain.
No. 13. Studded with 20 lbs. of three-penny nails, and covered on one side with zinc.

GROUP No. 7. PART OF SPRUCE PILE, 7 INS. DIAMETER, 36 INS. LONG.

No. 16. Studded with 4 lbs. of carpet tacks.
No. 17. " " 5 lbs. of three-penny nails.
In both cases driven from ½ to ¾ in. apart.

Group.	No. in group.	Description.	May, 1879.	May, 1880.	May, 1881.	May, 1882.	May, 1883.	Cost per square foot for protection.
No. 1.....	1	Spruce. No nails	Large number of worms.	Eaten more than in 1879.	Eaten more than in 1880.	Eaten more than in 1881.	Honey-combed.	
	2	Spruce, Carpet tacks.	No worms.	No worms.	No worms.	No worms.	No worms	72 cents.
	3	Spruce. Threepenny nails.	"	"	"	"	"	38 "
No. 4.....	10	Oak. No nails.	Large number of worms.	Large increase from 1879.	About same as in 1880.	Eaten more than in 1881.	Honey-combed.	
	11	Oak. Carpet tacks.	No worms.	No worms.	No worms.	No worms.	No worms.	80 cents.
No. 5.....	14	Pine. No nails.	30 to 40 worms.	50 to 60 worms.	Honey-combed.	Not as bad as in 1881.	About 180 worms	
	12	Pine. Bellows' nails and tacks.	No worms.	No worms.	No worms.	No worms.	No worms.	$1.00.
No. 6.....	15	Spruce. No nails.	30 to 40 worms.	60 to 70 worms.	Honey-combed.	Honey-combed.	Honey-combed.	
	13	Spruce. Threepenny nails.	No worms.	No worms.	No worms.	1 worm.	No worms.	43 cents.
No. 7.....	16	Spruce. Tacks.	1 worm.	1 worm.	1 worm.	1 worm.	No worms.	12 "
	17	Spruce. Threepenny nails.	3 or 4 worms.	9 or 10 worms.	About 17 worms.	About 17 worms.	10 worms.	10 "

Richard Lamb, Assoc. M. Am. Soc. C. E.—I think that the *teredo navalis* is somewhat of an epicure, and is not only looking for a home, but for something that it likes to eat. It seems to want wood that is acid. The charring of a pile gives a carbon coating that is not acid, and this it will not eat. If you leave the bark on the pile, the teredo will not attack it. There does not seem to be acid enough in bark to satisfy it. The place on the pile where the teredo will first attack will invariably be at the points where the charred wood or bark has been knocked off. The teredo chooses the woods that have the most acid in them. It will attack oak and pine before it will juniper or cypress, and it is generally known that it will scarcely ever attack the palmetto tree at all, which probably has less acid than any other wood.

I am somewhat interested in the Carolina Oil and Creosote Company. The experiments which the gentleman spoke of were made, and other experiments also in the same line. Ingredients of dead oil of coal tar were separated, and pieces of wood of similar size and texture were treated with each chemical extracted from the dead oil of coal tar. These pieces of treated wood were exposed at a place where the teredo is as ravenous as anywhere on the coast, and inside of one year every ingredient, except naphthaline, had been attacked. That seems to show that what we call "creosoting" is probably a misnomer, as the creosote itself was attacked, and the only chemical not attacked was the naphthaline.

Some of the gentlemen here who are interested in creosoting works know that when you buy dead oil of coal tar, if you specify the ingredients there will be a great difference in the price of one make of oil over another. I have noted a greater change for that containing an extra proportion of naphthaline. I think American engineers make a very great mistake in not specifying what their dead oil of coal tar shall contain. They call for "dead oil of coal tar," and there are just as many different kinds of dead oil of coal tar as there are gas houses. Most engineers seem to think that a piece of wood that will smell badly and look nasty is what they want. The best creosoted wood is of a dark leadish color, and the gummy substance on the surface of the wood which some engineers desire, and which I have reason to think some creosoting companies put on artificially, is not the effective treatment, as it after awhile leaves the pile by the action of the water. In the report of the American Society of Civil Engineers on the subject of wood preservatives, Mr. Andrews stated that he thought this gummy substance kept the worms from eating or boring into piles and was probably one of the main causes of keeping out the teredo. If this is the case my observation has shown me that the "main cause of keeping out the teredo" is removed in about one year. If creosoting is to stand on its merits it must be on the proper chemical properly injected into

the wood, and not upon a gummy substance which is soon washed off.

I made some experiments in filling the pores of wood with a material that would keep the teredo out, and that could not be washed off or dissolved by water. First I examined the teredo and found that the auger-shaped head with which it bores into the wood has a silicon point on it, so that it bores through the pile with glass. This suggested preventing its going into the wood by injecting glass into the pores of the wood. On reading up the subject of glass, I found that in England the walls of an old stone cathedral had been preserved from disintegrating, by injecting into the stone a solution of calcium chloride and silicate of soda (water glass). I succeeded in getting glass crystals into the pores of wood while at Norfolk, Va. The sample was exposed to the teredo, and in one year they had not touched it. Unfortunately I left the city, and the sample was subsequently lost. I have not since had time to experiment. I noted that after filling the pores with glass the wood did not increase in weight when soaked in water, showing that the wood did not absorb the water and would therefore be prevented from rotting. I also noted that the wood could not be burned. It would char, but not burn. It turned or dulled the edge of a blade when trying to cut it with a knife.

I think calcium chloride and water glass will probably be the chemicals with which the teredo will be kept out in the future. A little carbon mixed with it will make the glass very hard. He will be kept out mechanically by confronting him with glass harder than his glass auger, and not by poisoning him. By the use of calcium in this preparation the acid of the wood is also neutralized, making it less tempting as food for the teredo.

The teredo is a mysterious mollusk. If two planks are nailed together it will bore into one, and although it many come within $\frac{1}{16}$ in. of the other plank, it will not enter it, but will make its course often for many inches parallel with and exceedingly near the board it did not enter. Is there any gentleman here that can explain how the teredo grows in bulk? It attaches itself to a pile a small spawn no larger than a pin's head, and bores many inches into a pile, and at all times has its hole completely lined with a firm calcareous substance. The first year its diameter may not be over $\frac{1}{16}$ in., but the entire length of its self-made tunnel will be lined with a shell. The next year the entire length of the same tunnel may be twice the diameter and still lined with a correspondingly large shell. The consistency of the teredo is about that of an oyster. How can you explain its driving its auger into a hard oak pile, with no harder muscle than a snail's body? On still summer nights I have heard them grinding their way into the wood, and the noise of their grinding would surprise you if you should put your ear to the head of a pile in which they were at work.

JOHN A. MCDONALD, M. Am. Soc. C. E.—Mr. Thomson was saying that the teredo here attained $\frac{5}{8}$ in. in diameter; he does not give the length. In Australia we frequently find them $\frac{1}{2}$ in. in diameter, and the maximum length is about 3 ft.; a few years ago I took out two specimens of the teredo about 2 ft. long.

In regard to the teredo liking the timber, and in regard to its going into the timber to make a home there, there is generally some sense in the instincts of animals, and while the teredo will go through a small nail hole and will find its way through very small holes, yet it must know that it must grow and won't be able to come out again, and I take it is making the teredo out to be a bit more foolish than we ought to. We ought to think that the teredo goes in because it likes the timber, and it is willing to go in and spend the rest of its life feeding on the timber. The timber we use chiefly for piles weighs from 70 to 80 lbs. per cubic foot. As to the sample that Mr. Thomson passed around, I have seen in some of our northern rivers piles attacked as badly as that in 12 months' time. The accuracy with which the teredo will travel along a plank is wonderful; it will travel for yards close to a seam and within about $\frac{1}{16}$ in. of the seam and never go through it.

In regard to treating the piles, we never use creosote in Australian hard woods, because we cannot get the creosote into the woods; they are too dense; and we have to use external means. We generally use metal, and then replace any sticks which are attacked by the teredo. We have used the two modes described in the paper, that is, the iron tube and filling with concrete, and earthenware pipes.

H. COMER, Assoc. Am. Soc. C. E.—I merely wish to say that the two samples on the table were given to me some time ago; they are specimens of treated and untreated piles used in a railroad trestle and driven alongside each other for 15 or 16 years. The creosoting was done by Mr. Andrews, and I think they speak for themselves as to whether creosoting can prevent attacks from the teredo or not. I believe I am right when I say that when the American Society was making the investigations as regards the preservation of timber some years ago, it found that wherever treated or creosoted lumber had failed, and it could be traced out, it was proved to be through improper treatment, or a poor quality of oil had been used. The same result has been reached by our English friends. This, I am aware of, from pamphlets and different records I have read. I sent these specimens up to-night with the idea that they might be interesting to some of the members.

One of the members to-night said, in regard to specifications for dead oil, that that oil which had the most naphthaline in it was the most expensive. I can speak from personal experience, that it is the cheapest. The most expensive oil as used in creosoting on the other side is what is known as "country oil"; the country oil has a higher

percentage of tar acids and a lower percentage of naphthaline and is much more expensive. Mr. Bolton in his paper claimed that the combination of tar acids and naphthaline is what is so effective in preserving timber, and in his opinion the best oil was that which contained a high percentage of naphthaline.

H. W. Brinckerhoff, M. Am. Soc. C. E.—In regard to the question that has been raised as to whether the teredo goes into the pile for board or lodging, if the hole that it makes and occupies is very much larger than its own person, we might suppose that it has enlarged it for food and not merely for exercise. I would like to ask if any one has observed what proportion of the length of the tunnel made by the teredo is usually occupied by its body?

F. Collingwood, M. Am. Soc. C. E.—While connected with the construction of the dry dock at Newport News, located near the mouth of the James River, I was obliged to consider the subject of the protection of timber against the attacks of the *teredo navalis*, as the ravages of the worm are very extensive in those waters. A pile 1 ft. in diameter would be rendered useless for supporting heavy weights in the course of six months. Unfortunately the contract for the work had been signed before I was called upon, and the specifications required simply that the timber should be creosoted, but with no provision as to quality. The contractor had, therefore, arranged to have the creosoting done at Wilmington by the injection of wood creosote, which was prepared at the works by the distillation of yellow pine timber.

I made a personal visit to Wilmington and examined some small timbers which had been used as piles in Smith's Sound. These had been partially protected by the injection of the wood creosote oil, and had been in use a year. I found that while the protection was perfect, from high tide to some distance below, the worm had entered extensively near the mud line, and some timbers at another point had been eaten off. I brought back with me some of the oil, and requested an analysis, but it was never made.

At the coal docks of the Chesapeake and Ohio Railroad, at Newport News, some piles had been in use several years, which had been creosoted with dead oil creosote, after being seasoned. The creosoting was done by placing the piles in a tank containing the creosote, where they were soaked for about six weeks. These were not examined by a diver, but an examination at low tide showed no evidence of the entry of the teredo. It will be remembered that the English writer, Mr. Bolton, claims that the valuable principle in the dead oil creosote is especially the acridine; and on tasting the wood to which this creosote had been applied, it was exceedingly bitter, while the wood to which the wood creosote had been applied was sweetish in taste and almost devoid of bitterness. As the contract was allowed to be completed by

the use of wood creosote (which had received the endorsement of a board of officers appointed by the United States Government), I had an opportunity to see how it withstood the test. Upon examining the timber after it had been in place about a year, I found that the teredo had already begun to work on the angles of sticks near low water; and since that time extensive renewals have been necessary.

In reference to the question of the life habits of the teredo which has been under discussion this evening, I believe that it begins its life as a free swimmer, the same as the spat of the oyster, microscopic in size; it finally attaches itself to any wood surface and immediately begins boring and growing in length and diameter as it bores. The teredo has an influent and an effluent siphon, by means of which it takes in and discharges water, from which it abstracts the animalcules which form its food, and the carbonate of lime, with which it lines the hole it excavates. The original opening is never sealed, as the animal must have access to clean water, and dies as soon as it is surrounded by that which is stagnant or foul. There is no evidence that the wood excavated becomes a part of its food. The animal belongs to the class of borers, some of which bore into mud, some into shell, and some into stone, which certainly could not form food for the borer.

Charles Macdonald, M. Am. Soc. C. E.—The paper and discussion furnish the best evidence that timber piles are not suitable material for permanent construction in "teredo" waters. It is a question whether it would not be even better policy to adopt iron construction at once, where from motives of economy, a low first cost is a necessity. This might be either in the composite form used at Old Point Comfort, and described by John B. Duncklee, M. Am. Soc. C. E., in the *Transactions* of this Society, Vol. XXVII, page 115, or in solid metal throughout.

In 1883 I built an iron pier at the Island of Oruba, South America, using solid iron piles, 5 ins. in diameter.

They were easily driven to a firm bearing in the coral formation of the sea bottom, and have thus far given satisfactory results.

L. J. LeConte, M. Am. Soc. C. E. (by letter).—The results described in the paper agree very well with experience on the Pacific Coast harbors during the past 24 years. A great many attempts have been made towards creosoting piles as a protection against the teredo, but, generally speaking, the products put on the market have been very inferior, and consequently the piles soon became honey-combed. Of late, however, some good work has been done, and there are good reasons to believe that the future will develop a better record for creosoting work. Recent experience shows that in attempting to impregnate freshly cut or green timber with large percentages of dead oil, with the view of insuring perfect protection, another grave difficulty has presented itself. When the impregnation process is completed and the piles are hauled out from the boiler tanks, they are found to be

badly checked, some of the cracks ½ in. wide running nearly the length of the piles.

The sap wood also seems to be separated from the hard wood, so that when one kicks the pile it emits a hollow sound, similar to that of a rotten log. To prevent these cracks from running and opening during the process of driving, it becomes necessary to wrap the piles with telegraph wire, say six or eight round turns, at intervals of 6 ft. along the length of the pile. If they are to be driven in hard ground and this protection is omitted, the creosoted piles will splinter and break all to pieces under the blows of the pile-hammer. The life and strength of the timber seems to have departed. If any member can tell us how to creosote freshly cut green wood thoroughly, and at the same time not kill the life and strength of the fiber, we would be pleased to hear from him. The plans of using concrete and vitrified pipe for pile covering were both applied on works in San Francisco Bay some 20 years ago, with the same end in view, with this slight difference that the creosote was in the form of ordinary cement sewer pipe. The city front of San Francisco is exposed to very heavy storms from the southeast during the winter, and these coatings have failed to stand the shock due to driftwood driven by heavy seas. In sheltered places, doubtless, this is a good and lasting protection.

A still cheaper process is now being extensively used on the Pacific Coast for protecting piles between the mud line and high water-mark, namely, the Paraffine Paint Company's coating, consisting of a heavy coat of paint, then a jacket of heavy canvas saturated with the same paint and nailed on with copper-plated nails, and, finally, an outside protection of closely laid wooden battens, fastened on with wire nails. The latter is simply to protect the coating during the rough handling incident to pile-driving work in general. This process does not cost more than 35 to 40 cents per running foot of pile between the mud line and high tide, and has stood a severe test for eight years or more in waters where teredo and limnoria destroy ordinary piles and woodwork in from four to six months.

Another plan which has been extensively used on the Pacific Coast since 1867, and possibly before, consists in coating the timber with boiled coal tar; then a good covering of stout felting saturated with coal tar and fastened on with galvanized iron nails; and, finally, an outside protection of redwood battens or boards, as the case may require. The best test of this plan is that furnished by the facts connected with the construction of the United States Engineer's wharf at Lime Point, in the bay of San Francisco, near the Golden Gate. The foundations of this wharf consisted of Oregon fir cribs filled with stone. The timbers were square, the ends halved in and drift-bolted, so that the outside faces of the cribwork were tight jointed, smooth and continuous. These cribs were given an outside coating of coal tar and felting, as

above described, and finally sheathed with redwood boards. This work was done by Colonel George H. Mendell, Corps of Engineers, U. S. A., in the winter of 1867–68, and the woodwork is as sound now as the day it was put in, some 25 years ago.

The teredo and limnoria are very destructive in this locality, ordinary piles lasting five to six months only. This coating is now extensively used for wooden buoys, also to protect the bottoms of bay schooners, transfer lighters, dredging machines, etc.; in fact, is generally used for protection of all woodwork against the ravages of the teredo and limnoria instead of a covering of copper or composition metal.

The cost of this covering is very reasonable, say 5 cents per square foot all told. Some shipyards are now using heavy tarred roofing paper, instead of the more expensive felting above mentioned, and speak well of it; but sufficient time has not elapsed to place it on the same footing with good heavy felting.

Ernest Pontzen, Cor. M. Am. Soc. C. E. (by letter).—For many years I have been interested in the question of the preservation of timber. There are two methods of protecting timber against the *teredo navalis*, namely:

First.—Prevent the *teredo navalis* from approaching the timber.

Second.—Introduce into the timber poisons which will kill the teredo.

These methods can be used simultaneously. The introduction of concrete into the space between the piles and iron shells, and the use of vitrified clay pipe filled up with sand, are both methods under the first head. Impregnation with creosote or sulphate of copper comes under the second head, but the poisons, in order to produce the result required and to kill the *teredo navalis*, must be soluble; therefore, only soluble matters, which may be washed out by water, can be efficacious in the preservation of timber.

In order to prevent this washing out, it is well to combine the two methods of protection. The statement made in the paper under discussion, that "creosoted timber has been preserved from 1876 to 1886," proves that the useful parts of the creosote have been retained ten years without an outside coating to prevent washing out. In my opinion, it is quite sufficient to cover the creosoted wood with a thin plate of metal, or any other coating, in order to prevent washing out

of the soluble antiseptics. The use of a cement or concrete coating, as well as the use of vitrified clay pipes, appears to me to be insufficient when the timber has not been treated with poisons which will kill the teredo, because the finest cracks allow entrance to this very small animal. If the timber, however, has been treated with antiseptics, a less extensive outside protection to prevent the washing out can be provided.

If the timber has been treated with sulphate of copper, I think that the exposed parts could be subjected to electric action, which would produce galvanically a thin coating of copper. The timber can also be subjected after treatment with sulphate of copper to the successive action of a silicate or of sulphate of barium, producing on the surface a non-soluble combination. If the timber has been treated with creosote, I think it possible to cover it with layers of tar and sand which would prevent washing out.

As the parts of the timber exposed to the attack of the *teredo navalis* are those above the mud and between high and low water, there is no danger that this coating will be injured in the driving of the pile.

These ideas have not been sustained by practical tests, but it is my opinion that they could be, and I submit them to the judgment of the eminent engineers who are interested in the preservation of timber.

R. Montfort, M. Am. Soc. C. E.—The piles used on the Louisville and Nashville Railroad Company's lines were round, with the bark removed prior to treatment. The sap was allowed to remain on, as it absorbs a larger proportion of oil under pressure than is possible to force into the heart. When the bark is left on untreated piles it protects them from the teredo for a short time, but within two or three years they are destroyed. Charring would doubtless protect them in a similar manner.

The oil used in treating piles on the New Orleans and Mobile Division was manufactured in this country; the oil used in treating the wharves at Pensacola, Fla., was imported from England. The American oil has given better results. Samples of piles that had been attacked by the teredo were sent to Mr. S. P. Boulton, of London, Eng., in 1887, for inspection and analyses. His general conclusions were that the weight of the oil used (8¾ lbs. to the gallon) was not sufficient, and that oils of a heavier description and higher distilling points, rich in

green oil and the less volatile antiseptics, would have given better results.

After separating the oils from the wood resin, with which they had become incorporated, no less than 95% came over on distillation up to 600° Fahr. The lighter antiseptics are unreliable from their washing out or evaporating. He also considered the trouble, in a great measure, due to the limited time for seasoning the timber, which resulted in insufficient removal of the moisture before creosoting.

The fact that the piles resisted the attack of the teredo for from six to 14 years, and that 952, out of a total of 4 274, showed no signs of attack at the end of 14 years, would seem to prove conclusively that the oil used must originally have contained the proper constituent to protect against the teredo, and that the action of the waves gradually washed it out, thus allowing the teredo to attack the piles.

In recent years, we have been experimenting with a mixture of resin and dead oil, with the hope that the resin will aid in fixing the oil and preventing it from being dissolved or washed out. Sufficient time has not elapsed to form any definite conclusion, although so far piles thus treated have given satisfactory results. There can be little question but that it is practicable to construct piles of iron or steel that will be more durable than wood treated with creosote or other known protections. As to whether the metal piles are cheaper in the long run, taking into consideration the difference in first cost, with compound interest on the saving effected by using wood, is something that must be settled independently in each case and cannot be decided at once for all cases.

At Bay St. Louis and Biloxi trestles the piles average 50 ft. or over in length, and cost 30 cents per foot, $15 per pile; driving, per pile, $3; capping and bracing, $2; total cost per pile in place, $20. If 50-ft. iron piles 5 ins. diameter, such as those used successfully by Mr. Charles Macdonald in 1883 in the construction of an iron pier at Island Oruba, South America, were substituted for the creosoted yellow-pine piles, they would weigh 65½ lbs. per linear foot, or a total of 3 275 lbs., and would probably cost about 3 cents per pound in place, equal to $98.25. If we add screw end and cap, say, 1 000 lbs., at 3 cents in place, $30; for bracing, struts and ties, $21.75 in place, makes the total cost per pile $150. Difference in first cost in favor of creosoted wooden pile, $130; compound interest on this $130 in three years at 6% would

equal $24.83, or more than sufficient to entirely renew the creosoted pile. The creosoted piles at Bay St. Louis and Biloxi have been in use for 14 years, and with exception of the small percentage that was destroyed in the hurricane of October 2d, 1893, will probably continue in use for at least 10 years to come. The cost of the recent protection has been much less than it would have been to have replaced the existing piles by new creosoted piles. Even if the cost of the 5-in. diameter solid iron or steel piles could be reduced to one-fourth of that indicated by the foregoing figures, we would still be unable to show economy in their use as compared with creosoted piles. The creosoted piles average 14 ins. in diameter above the bottom of the bay. If we figure their strength as columns compared with the 5-in. iron piles, assuming both piles braced so as to fix them at points 12 ft. apart, measured along the piles, the pine piles are twice as strong as the 5-in. wrought-iron piles, while their transverse strength is nearly three times as great to resist blows from drift. It is true that the bearing power of a wrought-iron pile as against settlement can be increased indefinitely by increasing the size of the screw disc on which it rests, but no trouble from settlement has been experienced with the pine piles.

At Mobile River, on the Mobile and Montgomery division of the Louisville and Nashville Railroad, a bridge consisting of five fixed spans, each 140 ft. in length, and a draw span 259 ft. long, rests upon wrought-iron screw pile piers driven in 1871. The piles are made solid, 6 ins. diameter, and the construction of the piers is fully described in the *Engineering News and American Contract Journal* of April 4th, 1885, in an article by Col. Wm. M. Patton, C. E. The piles have done good service and are still in use. It has, however, long since become necessary to drive wooden piles in front of the piers and construct fenders to protect them against blows from the large drift that floats in the river. Recently, when rebuilding the superstructure of the bridge, the piers were strengthened. As to what the life of 5-in. solid wrought-iron or steel piles at Bay St. Louis and Biloxi would be is difficult to say, but creosoted pile piers at West Pascagoula and Pearl River, where the water is not nearly so salt, were originally braced with 1¼-in. round rods; many of these rusted completely off within ten years. Also a ⅜-in. wrought-iron drum on the center pier of Biloxi draw rusted entirely through.

The *Engineering News* of February 8th, 1894, contains a very interesting article on teredo-proof sheathing for piles used on the Pacific Coast, and a report of Mr. Howard C. Holmes, Chief Engineer of the California Board of Harbor Commissioners. Among others, he describes the method used by A. W. von Schmidt, which consisted of a loose jacket of earthen sewer pipe, with the joints and interior space filled with hydraulic cement. This did not give good results, on account of blows from drift and cracking, due to want of elasticity. The pipe at Bay St. Louis and Biloxi is extra strong, and the joints are made with a composition of coal-tar pitch and sand, which is more elastic than cement. The space between pipe and pile is filled with sand. Drift is practically unknown, except in rare cases of extraordinary hurricanes. Mr. Holmes also describes methods of coating the piles with asphalt, marine cement and Portland cement, applied with a brush, three coats before driving. None of these methods at all correspond to those used by the Louisville and Nashville Railroad Company.

The Paraffine Paint Company's process and the coal-tar process used at Lime Point, described by Mr. L. J. Le Conte, are not applicable to piles already driven and which have been in use for a number of years, such as those in Bay St. Louis and Biloxi trestles.

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694.

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THE TOWER OF THE NEW CITY HALL AT PHILADELPHIA, PA.

By C. R. GRIMM, M. Am. Soc. C. E.

READ NOVEMBER 15TH, 1893.

WITH DISCUSSION.

Volumes innumerable have been written treating of the theory, design and construction of plane trusses, trusses whose members have their axes lying in one and the same plane; and it has long been the practice to construct domes, conical roofs, etc., supported by a number of such trusses occupying planes radial to the vertical axis of the structure. J. W. Schwedler conceived the idea of designing the wrought-iron "mantle construction," which bears his name, in which the stresses occur and are taken up in the mantle of the structure, leaving all the interior space unoccupied by trusses.

The literature of trusses having any position in space (German, *räumliches Fachwerk*) is as yet very scanty, and a general and exhaustive theory of such trusses has yet to be written. Hence, and inasmuch as the wrought-iron tower framework surmounting the masonry tower of the new City Hall at Philadelphia is a construction

of this kind, the writer believes that no apology is necessary for presenting the following account to this Society.

General Description of the Work.—Up to a height of 337 ft. $4\frac{1}{2}$ ins. from the street level the tower of the City Hall is of brick, faced with marble, and this portion is now practically completed. In order to avoid excessive pressure upon the foundations, the commission in charge of the erection of the building decided to construct the remaining portion (about 173 ft. in height) of metal, and the contract for this, complete, including statuary, was awarded to the Tacony Iron and Metal Company, at Tacony, Philadelphia. The architectural design of the tower, shown on Plate XXXI, furnished the sole basis for the engineering design, and was the work of the late Mr. John McArthur, the architect of the building.

The tower was placed in charge of the writer by the Tacony Iron and Metal Company, who designed the engineering work in general and in detail.

The framework of the entire metallic structure (Plate XXXII), resting upon granite blocks, consists essentially of an octagonal prism surmounted by an octagonal pyramid of the same diameter at base as that of the prism. Throughout the entire height of the prism and pyramid their horizontal sections form regular octagons. This wrought-iron framework may be divided into a primary and a secondary construction. The former constitutes the true skeleton of the structure, while the latter, immediately surrounding the primary construction, forms the direct support of the shell, and transmits to the primary construction the wind pressure upon it, and, in the case of the pyramid, its weight also. In the case of the prismatic portion, the weight of the shell is carried by the secondary construction directly to the masonry of the tower. The octagon of the prism passing through the centers of the eight columns of the primary construction has an inside diameter of 49 ft. $1\frac{1}{8}$ ins., and an outside diameter of 53 ft. $1\frac{21}{32}$ ins.; the octagon at the top of the pyramid has an inside diameter of 4 ft. 6 ins., and an outside diameter of 4 ft. $10\frac{1}{2}$ ins. The height from the top of the granite to the center of the connection between the prism and the pyramid measures 67 ft. $4\frac{1}{8}$ ins., and the height from the latter point to the top of the tower-cap is 104 ft. $3\frac{7}{8}$ ins. The tower-cap will receive the colossal bronze statue of William Penn, which is 37 ft. high, and as the granite level is 1 ft. 7 ins. above the level of the marble, we have

a total height from the ground to the top of the statue of William Penn of 547 ft. 7½ ins. Other main dimensions can readily be taken from the general design.

The primary construction is divided into ten stories, three for the prism, and seven for the pyramid; and it consists of the eight rafters forming the corners of the pyramid, and of the eight columns forming those of the prism, together with the octagonal braced rings and the diagonals. A panel of this "truss in space" is comprised between two rafter sections of the pyramid (or two column sections of the prism), and two ring sections, one above the other. Each panel is crossed by two diagonals, as shown. These diagonals are designed for tension, and those in the three top stories for compression also.

The eight columns of the prism rest upon four wrought-iron box girders, marked *G* (Plate XXXII). These girders rest upon cast-iron wall plates 3 ft. wide, 11 ft. long and 2½ ns. thick. These wall plates are placed under the centers of the box girders, and, inasmuch as the latter are 24 ft. 4 ins. long from out to out, there is at each end a projecting arm, or cantilever, 6 ft. 8 ins. long. The columns of the prism are supported by these projecting arms 4 ft. 8 ins. from the ends of the wall plates. This arrangement is necessitated by the fact that only those portions of the masonry of the tower under the wall plates are so constructed as to bear the weight imposed.

Each of the wind bracings consists of two sets of horizontal tension members acting in connection with the compression members which form the octagonal ring at the top of each story; each set consists of four members forming a square. These members are square bars, except in the two uppermost stories, where they are of angle iron. Since the feet of the columns in the prism are anchored to the box girders, wind bracing is not required at that point. The octagonal ring has, however, been retained. Such horizontal structures permit only such deformations of the tower framework as are due to the elasticity of the material. The selection of this form of bracing was dictated by the necessity for a spiral staircase within the tower, reaching from the top of the stone tower up to the balcony. It is true that the bracings might have been placed on the outside of the primary construction, but practical considerations rendered such an arrangement undesirable.

The secondary construction of the prism consists essentially of

four skeleton columns, 56 ft. 9 ins. high, marked *P*, and four arches, each of 36 ft. 3 ins. span, marked *A*, together with three octagonal rings, and 76 vertical posts inserted between those rings.

The secondary construction of the pyramid consists essentially of octagonal rings, which support the jack rafters.

There are, in all, three floors and one balcony to be carried by the tower framework. The first floor is at the top of the prism; the second, about in the middle of the sixth story, and the third, at the top of the seventh story. A door in the shell of the tower leads from the third floor to the balcony.

It was originally intended to build the first floor of concrete laid upon corrugated iron arches placed between I beams ; but, in order to reduce the weight, it was decided to use steel plates and rubber. The cantilevers supporting the present floor were designed and built to carry a concrete floor. They are pin-connected trusses, and spring from the columns of the prism, to which they are attached by pins. The top and bottom chords of these cantilevers are made each of two channels, connected by plates ; the posts are composed of four angles and one web, and the diagonals are square bars. A stiff ring, made up of plates and angles, unites the ends of the eight trusses, leaving in the center a vacant space of about 12 ft. in diameter. The thrust of the bottom chords of the cantilevers, which would produce a bending moment on the columns, is taken up by eight tie rods running around the prism. By means of pins these tie rods are connected with bent plates attached to the outer sides of the columns, and, to prevent buckling of the angles of the columns, two stiffeners are riveted on each side of the latter. Although the feet of the pyramid rafters are connected with each other by struts, to which the diagonals are attached, tie rods, with such details as just given, intended to take the outward thrust caused by the pyramid, are placed around the top of the prism. The feet of the rafters of the pyramid are centrally connected with the heads of the columns of the prism by means of pins $4\frac{11}{16}$ ins. in finished diameter. These pins are in the same horizontal plane with the tie rods just referred to. The wind bracing at the top of the prism is placed as near as practicable to the top chords of the cantilevers. A horizontal structure of the same description has also been inserted, as near as possible to the level of the second floor, to avoid the bending moment exerted by the upper chord of the can-

tilevers of this floor upon the rafters, and tending to bend them inward. From this floor a view may be had through the lunette windows. The supporting parts of the second floor are made of angles and plates. The wind bracing at the top of the seventh story is also placed as close as practicable to the level of the floor. The rafters of the seventh story are built as trusses. They carry the third floor and the balcony, and during erection a false work from 75 to 80 ft. high, to be used in the mounting of the statue of William Penn.

The several floors of the tower are composed of I beams bolted directly to the brackets which support them. Steel plates, $\frac{1}{4}$ in. thick, are secured to the tops of the I beams by means of bolts having countersunk heads, and a layer of rubber about $\frac{1}{2}$ in. thick will form the floor surface. The only exception to this is the floor of the balcony, which will be finished with a layer of asphalt. This latter floor is several inches lower than the adjacent third floor in order to prevent rain water from entering the tower.

Brackets attached to the rafters of the pyramid, and consisting of two horizontal and two inclined angles, carry at their extremities struts, which in turn carry the jack rafters. To the latter the shell is fastened by light cast-iron brackets, as already observed. At the center of each jack rafter are two lattice bars, one running from the top angle to the rear of the strut, and the other from the bottom angle to the same point. They are designed to prevent buckling of the angles in front of the strut.

It will be seen from the plan, Plate XXXII, that on each of four sides of the tower—each two sides being opposite—and just outside of the prism, are two pairs of cast-iron columns, indicated by dotted lines, and behind each pair of columns stands a pair of pilasters, also indicated by dotted lines. The columns are 2 ft. 8 ins. in diameter at bottom, tapering to 2 ft. 4 ins. at top, and the width of the pilasters is about the same, with a depth of 15 ins. The metal in both columns and pilasters is about 1 in. thick, and their height is 37 ft. Each pair of columns, with the corresponding pair of pilasters, is bolted to the masonry, and the four are united at the top by a heavy cast-iron plate. On these plates rest the feet of wrought-iron arches of 36 ft. 3-in. spans (see Plate XXXII) having a uniform depth of 3 ft. 6 ins. and a width of 3 ft. $5\frac{1}{4}$ ins. The inclined members running from the second panel point at each end to the points of support are

designed to add to the rigidity of the arch. These arches are built of plates and angles. Each end is held down by three 1¼-in. bolts, and free to slide upon the planed surface of the supporting plate. The load on an arch is about equally distributed, so that the overturning moment is inconsiderable. To meet this moment, however, the crown of the arch is anchored by a plate 14½ ins. wide, 5 ft. 6 ins. long, and ½ in. thick, to the foot of a post, standing in the rear, and thus indirectly to the primary construction. Upon granite blocks at the four corners of the tower and just outside of the octagon of the prism stand four wrought-iron skeleton columns, bolted down to the blocks, each by eight 1¼-in. bolts; they are marked *P*, and are 56 ft. 9 ins. high, 5 ft. 6 ins. square, and built of angles and plates. Three octagonal rings, a part of the secondary construction, surround the prism. Each section of these rings is a box strut, 18 ins. square. Between the lowest and intermediate ring and between the latter and the upper ring are inserted 76 wrought-iron vertical posts, each built of four angles, I-shaped, latticed and spaced not more than 4 ft. apart. To these posts the upper part of the shell is fastened by means of bolts and cast-iron brackets. These rings are placed between the primary construction on the inside, and the arches and the skeleton columns on the outside. Those ring sections which pass the rear of the arches are fastened to and carried by them, and consequently the posts and the ring sections, lying all in the same vertical plane, together with the upper parts of the shell of the prism on these sides, are finally carried by the wrought-iron arches which transfer the dead weight through the columns and pilasters to the granite. To the outer faces of the arches are attached triangular brackets carrying the pediments. To the bottom of the arch is attached an ornament representing branches with foliage. This is 26 ft. in length and extends in a vertical plane parallel with the side of the prism and nearly under the center line of the arch. On the crowns of the arches are placed wrought-iron boxes to receive the bronze eagles. The writer carried out the same idea, as far as the distribution of dead weight is concerned, on the remaining four sides with the skeleton columns. On these four sides the vertical posts between the lowest and intermediate rings are omitted as superfluous. As will be seen from the diagrams, the ring sections are carried by brackets on the inner faces of the columns, and the outer faces of the latter carry, partly by wrought-iron and

partly by cast-iron brackets, the larger part of the corners of the shell. The weight of the shell of the prismatic or lower portion of the structure is thus transferred, as already remarked, by the arches and cast-iron columns and pilasters and by the skeleton columns directly to the masonry, independently of the primary construction. Each of the four skeleton columns is to carry a statue 24 ft. in height. These statues have cast-iron ground-plates. They represent respectively an Indian with a dog, a squaw with a child, a Swede with a child, and a Swedish woman with a lamb. On top of the cast-iron columns is a light wrought-iron framework to carry ornaments. Cast-iron brackets for shell attachment are spaced 4 ft. apart as a maximum.

Although, as already explained, the weight of the prism shell is carried to the masonry independently of the primary construction, its wind stresses are carried to that construction by horizontally rigid connections composed of angle-iron brackets and bolts.

The eight columns resting upon the box girders are each bolted down by five $1\frac{3}{8}$-in. round bolts, and, as a further anchorage, two $1\frac{1}{4}$-in. square rods will be used for each column. These square rods have open turn-buckles and run across the tops of the channels and the bottom of the box girders. Each of the four supporting girders is secured to the masonry by two anchor bolts, 3 ins. in diameter and extending through the entire depth of the girders and 50 ft. into the masonry. These bolts are in close neighborhood of cross-plates between the webs of the girders and pass through slotted holes, thus allowing for expansion and contraction under changes of temperature.

The tower-cap is in two pieces, as is also the bronze base-plate for the Penn statue. The splice lines of the tower-cap and base-plate are at right angles to each other. The connection between the statue and the tower is effected by 10 2-in. round steel anchor bolts with the nuts inside the feet, the bronze plate acting as filling piece. The image of Penn and the tree trunk upon which Penn's hand rests are connected by 50 $\frac{3}{4}$-in. bolts, the center of this connection being about 16 ft. from the top of the ground-plate; 36 1-in. bolts attach the tree trunk to the bronze plate, and eight 2-in. round steel eye-bolts with 3-in. turned pins, passing through double brackets, connect the bronze plate with the tower.

All rafters of the pyramid and all columns of the prism have two web plates and four angles latticed, and only in the case of the

bottom columns a cover plate has been added. The tie rods of the wind bracing are pin-connected; the lateral details consist of two plates and four angles. These plates and angles form a diaphragm and protect the columns and rafters from the thrust of the rings. The eight struts, forming one of the octagonal rings of the primary construction, are made of four angles. These struts are box-shaped for the prism and latticed on all four sides; for the pyramid they are I-shaped and latticed, while the two uppermost rings have a solid web. The diagonals are pin-connected to the struts. All tension members have either clevises or single eyes with open turn-buckles. The jack rafters are made of two angles, curved to a radius of 136 ft. 2 ins., and trussed. The main and lateral pins are of steel, turned, and the pin-holes are $\frac{1}{64}$ in. larger in diameter than the pins.

It has been the aim of the writer to design all details centrally, and in case a bending moment is thrown on a member, in addition to the stress due to its position as a member of the structure, such member has been proportioned to resist both.

The tower-shell is of cast iron, except the louvres of the pyramid where steel plates have been used $\frac{1}{8}$ in. thick. This material was selected for the louvres in order to save weight. The thickness of the shell for the prism is from $\frac{5}{16}$ in. to $\frac{3}{8}$ in., and that for the pyramid from $\frac{1}{4}$ in. to $\frac{5}{16}$ in. The lower sections of the eight sides of the pyramid, reaching nearly up to the balcony, form parts of a cylinder with a radius of 137 ft. 6 ins. The four clock dials are placed below the arches and have a diameter of about 22 ft. They will probably be illuminated by electric light.

It is not considered necessary to go into a minute description of the shell, as the general plan shows it in its outlines.

All wrought-iron and cast-iron work for the tower was nearly finished in 1892, and the wrought-iron framework for the prism has already been set in place.

Plating.—In order to avoid the great expense involved in keeping the iron-work painted and free from rust, the tower-shell will be electro-plated with copper; but as this metal turns green and becomes unsightly, it is given a finishing coat of aluminum, which harmonizes well in color with the rest of the tower. The separate pieces are first boiled in a strong solution of caustic soda and then washed with water from a hose, whereupon they are pickled with dilute sulphuric acid until

all the rust is dissolved. They are next thoroughly cleaned with steel brushes and water, and are then ready for plating. Inside surfaces, which do not require any plating, are protected by paraffine wax. The total thickness of the metallic coat is from $\frac{1}{32}$ to $\frac{1}{16}$ in. Mr. J. D. Darling was placed in charge of the plating.

Statuary Work.—All this work, as mentioned before, is of bronze, composed of 88 parts of copper, 10 parts of tin, and 2 parts of zinc. The several pieces forming a single figure or group are rigidly united by means of flanges about 3½ ins. wide and ¾-in. and ⅞-in. bolts, spaced not more than about 6 ins. apart. The metal averages about ⅜ in. thick, but in the lower extremities of the figures it is made considerably heavier. No inside braces are used. The patterns for the foundry are of plaster of paris.

Erection.—The scaffolds to be used in the erection of the tower are of different design, and are now all completed, with the exception of that to be used in the erection of the Penn statue. This one has not yet been designed.

Owing to the complex nature of the structure, both the wrought-iron constructions and the shell complete were erected in sections in the company's yard. The first section is represented by the prism, the second by the four lower stories of the pyramid, up to the balcony, and the third by the rest of the tower. Generally speaking, the final erection is merely a repetition of the preliminary erection at the shops. The tower shell has been put together with open joints by inserting at short intervals small sheet-iron filling pieces of about the same thickness as the plating. In the final erection the shell will be made watertight by the use of usudurian. Brass bolts, with countersunk heads on the outside, will be used for the shell.

Since the top of the stone tower projects but a few feet beyond the base of the metallic prism, a working platform of timber (Plate XXXIV) has been provided at this level. This platform is in ground-plan a square of about 100-ft. sides, with the corners cut off. It serves, not only as a floor for the erecting gang, but also as a protection against damage to persons or property below from falling tools, etc. It is carried by 16 trussed flitch beams, acting as cantilevers. These beams are composed each of two timbers, 18 ins. by 7 ins. thick, having an iron plate between them bolted together by ⅞-in. bolts. Eight of these beams, as shown in the plan, are 50 ft. long, and the

others 42 ft. In the longer beams the 18-in. iron plates are 1 in. thick, in the shorter ones $\frac{7}{8}$ in. The beams are arranged radially, as shown, and their inner ends are joined by an iron ring 19 ft. in diameter. 32 $1\frac{3}{8}$-in. square vertical rods, two for each beam, take hold of a pair of plate girders (originally built for another purpose), by means of a framework of 10-in. I beams; but since these plate girders do not extend into the walls they are connected by vertical rods with another pair of plate girders, situated some 60 ft. below the top of the stone tower and built into the walls of the tower. The flitch beams are braced laterally, as shown, and in order to protect the platform against vertical vibrations the inner ends of the flitch beams are supported by wooden posts, resting upon the brick arches of the masonry of the tower, while guy lines extend from the outer ends of the flitch beams to the masonry of the tower below, where they are secured by adjustable bolts. The platform has an inclination of $\frac{1}{4}$ in. in 1 ft. to provide for drainage, and is enclosed by a very substantial railing about 8 ft. in height, provided with a wire netting. The dimensions can readily be taken from the drawing.

In erection, the steam hoisting machine used in constructing the masonry portion of the tower is employed; the end of its horizontal beam is directly over the center of a trap-door in the platform through which all working pieces pass before being distributed over the floor.

The wrought-iron prismatic construction, as already mentioned, is now in place. The scaffolding for erecting the shell of the prism will rest upon this platform. The lower sections of the shell, however, cannot be assembled until after the platform has been removed, because the flitch beams of the platform penetrate the sides of the prism. The four statues, which are to stand upon the skeleton columns at the corner of the tower, can readily be picked up in sections by a derrick standing on the first floor. In erecting the four lower stories of the pyramid a scaffolding is used. This will be carried by the eight cantilevers of the first floor. It carries at its top a revolving derrick, which is indicated in dotted lines on the general design. The tower shell will be put in place after the mounting of the statue of William Penn. When all of the wrought iron is assembled up to the balcony, the derrick will be removed and a scaffolding from 75 to 80 ft. high, shown by dotted lines in the general design, erected.

The balcony brackets, which form a part of the trussed rafters, have been designed to carry this scaffolding, each leg of which will be anchored down by two $1\frac{3}{4}$-in. round bolts. The scaffold is an eight-sided prism of 20 ft. outside diameter, and, since all the space inside of it is needed, the wind bracings should be placed on the outside of the scaffold. A hoisting apparatus will be placed on top of it, and the material raised will pass up inside the tower, the scaffold acting as a cage. In order to economize working space only six of the eight rafters of the pyramid will first be put in place, together with the tower-cap and the ground-plate for the statue, which will then be raised in sections. Finally, the remaining two rafters, the rings and the diagonals will be inserted.

Two light platforms of timber, 3 ft. wide and about 18 ft. long, suspended from the revolving derrick, have been used in putting together the shell of the pyramid, and these will again be employed in the final erection. The diagram (Plate XXXII) of the false work, used in erecting the pyramid, shows the horizontal timbers nearly on the same level with the primary rings.

This arrangement has been made to provide for working floors to be carried by the horizontal timbers and the rings. These floors, together with the suspended platforms, enable the erecting gang to reach any point of the shell, either inside or outside.

After the large platform and the steam hoisting machine have been removed, the bottom sections of the shell will be put in, and this will complete the work of erecting the tower.

Material.—The tower framework is of wrought iron. All pins and the anchor bolts for the statue of William Penn are of steel.

The shell is of cast iron, with the exception of the louvres, which are made of steel plates.

The statuary work is of bronze.

Loads.

Dead Loads.—The tower has been proportioned to carry the following loads, viz. :

The primary framework carries its own weight and that of three floors, the balcony, the secondary construction and shell of the pyramid and the statue of William Penn.

The secondary construction of the prism carries its own weight and

that of the greater part of the shell of the prism, four statues and four eagles.

The remainder of the shell of the prism is carried directly by the masonry.

Live Loads.—Each of the three floors and the balcony have been proportioned to carry a load of 100 lbs. per square foot.

Wind.—Wind stresses have been calculated for a pressure of 50 lbs. per square foot, supposed to be acting horizontally against the shell, the surface of which, for this purpose, is regarded as vertical throughout, while the prismatic part of the tower has been treated as a four-sided prism, whose width equals the full diameter from out to out of the shell.

In order to compensate for the wind-pressure on the tall scaffolding standing upon the balcony, the placing of the shell of the pyramid will be deferred until after the removal of the scaffolding, so that the wind stresses during erection will be less than after the completion of the structure. Hence, these stresses have not been taken into account. All other stresses due to erection have been considered.

Working Stresses.

The writer allowed the following limits for the stresses per square inch in pounds, for the different members:

	Lbs.
Rolled bars in tension	15 000
Plates and shapes	12 000
Compression members, whose lengths are less than 50 times their least radii of gyration	10 000
Shearing across fibers	7 500
Bearing	12 000
Bending stress on pins	20 000
Shearing	10 000
Bearing	15 000

The bearing surfaces of pins and rivets have been reckoned from the diameter.

Compression members, whose lengths exceed 50 times their least

radii of gyration, have been proportioned according to the following formula:

Flat ends.....$S = \dfrac{10\,000}{1 + \dfrac{l^2}{36\,000\,r^2}}$

One flat end and one pin end.....$S = \dfrac{10\,000}{1 + \dfrac{l^2}{24\,000\,r^2}}$

Pin ends.....$S = \dfrac{10\,000}{1 + \dfrac{l^2}{18\,000\,r^2}}$

where S = allowable stress per square inch,

l = length in inches of member between supports,

r = least radius of gyration of cross-section.

The intensities of the working stresses for such members as are subjected to alternate stresses, are:

For tension.....$12\,000 \left(1 - \dfrac{1 \text{ min. stress.}}{2 \text{ max. stress.}}\right)$

For compression members, whose lengths are less than 50 times their least radii of gyration, $10\,000 \left(1 - \dfrac{1 \text{ min. stress.}}{2 \text{ max. stress.}}\right)$

For compression members, whose lengths exceed 50 times their least radii of gyration, the numerators in the above expressions have been reduced to $10\,000 \left(1 - \dfrac{1 \text{ min. stress.}}{2 \text{ max. stress.}}\right)$

No distinction has been made between the dead load and the wind stresses as far as the intensities of working stresses are concerned.

The minimum thickness of wrought iron is $\frac{1}{4}$ in., except in the more important members, where it is $\frac{5}{16}$ in. Field riveting will be avoided entirely, since at many places it is difficult to reach the rivets, and riveting would be surely unsatisfactory. The more important connections, such as rafter splices, etc., will be made by means of turned bolts, which have been given a slight taper to insure a tight fit. Minor connections will be made by rough bolts.

All tension members have been subjected in a testing machine to a stress of 20 000 lbs. per square inch.

Painting.—The wrought-iron construction has been given two coats of oxide of iron paint, and all surfaces coming in contact were painted before being riveted together.

Approximate Method of Computations.

The assumed dead weights for the tower complete are grouped as follows:

(1) Weight of wrought-iron construction for each story of the tower, including the weight of the secondary construction for the pyramid, and excluding same for the prism.

(2) Total weight of secondary construction of the prism.

(3) Weight of tower-cap.

(4) Weight of supporting girders.

(5) Weight of shell for each story of the pyramid.

(6) Total weight of shell for the prism.

(7) Weights of statuary work.

(8) Live weights.

All the above weights are given in tons.

The assumed wind forces, applied at the different stories and at the statue on the top, are also given in tons.

Dead Load and Live Load Stresses.—The dead weight affects all the members of the framework, except the diagonals and the rings of the primary construction in the prism and the diagonals of the pyramid. The determination of these stresses, together with those due to live load, is governed by well-known laws.

Wind Stresses—Stresses in Rafters.—In determining the wind stresses, the tower is considered to be a cantilever, fixed at the bottom. Passing a section through the tower, we apply the stresses in the members thus cut to maintain the equilibrium and compute the wind moment. If we now assume that the stresses in the rafters resist this wind moment, we can write an equation between these stresses and the moment of the exterior forces. Assuming further that the stresses in the rafters vary as their distances from the axis around which the wind moment has been taken, expressing the stresses in those rafters nearest the axis in terms of those farthest removed from the axis, and substituting these values in the original equation, we can easily find the maximum compression and tension.

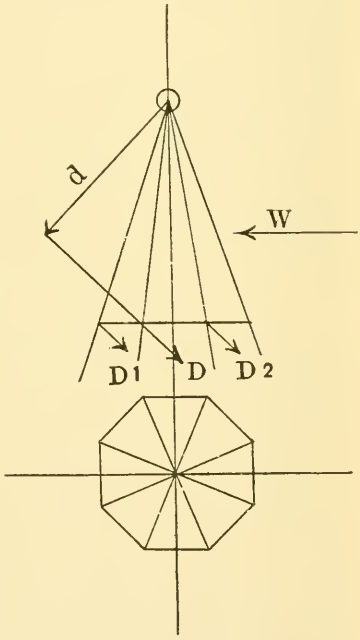
Stresses in Diagonals.—In computing the stresses in the diagonals we take as the center of moments the intersection point of the rafters and determine the wind moment around that center, which we will call *M*. With regard to the diagonals of the prism the center of moments is

removed to an infinite distance. The wind force will call into action only one system of diagonals, whose lever arms we denote by d (see sketch). The equation expressing equilibrium will then be:

$$M = 2D\,d + 2D_1\,d.\ \cos.\ 45° + 2D_2\,d.\ \cos.\ 45°:$$

In this equation the writer considered the sides of the pyramid as vertical.

Assuming now that the vertical components of the diagonals vary inversely as their distances from the axis of the tower, and expressing D_1 and D_2 in terms of D, we can find the stresses D in the diagonals from the above equation.



Stresses in Rings.—The rings take the horizontal components of the stresses in the diagonals.

Stresses in Wind Bracings.—If W denote the maximum wind force at any panel point, R the compression in a ring section; T the tension in a tie rod, we have—

$$R = \tfrac{1}{2}\ W \text{ sec. } 67\tfrac{1}{2}°, \text{ and}$$

$$T = \tfrac{1}{2}\ W \text{ tg. } 67\tfrac{1}{2}°.$$

Temperature Stresses.—Temperature stresses need not be considered, since the supporting box girders move on planed surfaces, and the construction is, of course, perfectly free to expand vertically upward.

The wrought and cast iron of the prismatic part of the tower have been weighed, and the assumed loads are found to agree fairly well with the actual weights, the former being but very slightly in excess.

Although all the wrought-iron and cast-iron work of the pyramid is done, none of the parts have thus far been weighed.

The estimated weight of 50 000 lbs. for the Penn statue proved nearly correct. The four statues on the skeleton columns have been assumed to weigh each 30 000 lbs., whereas the actual weight is only about 20 000 lbs. for each.

The total estimated weights for the tower are:

Wrought iron	376 tons.
Cast iron	519 "
Statuary work	105 "
Making a total of	1 000 "
The total wind pressure amounts to	183.7 tons.

Since the overturning moment of the wind is one-half that due to the dead weight, it follows that a pressure of about 100 lbs. per square foot would overturn the tower, were it not for the resistance by the anchor bolts.

The approximate area of the tower-shell is 57 000 sq. ft., and it is believed that the structure, when completed, will be the heaviest metallic tower ever erected at such an altitude.

The approximate method of computations was used in designing the tall tower pyramid of the St. Petri Church in Hamburg, Germany, built from data by Schwedler and finished in the year 1878.

Exact Method of Computations.

In designing a rigid structure with superfluous bars, we must ascertain whether the structure would remain rigid after the superfluous bars are removed. Let us, therefore, consider what is the criterion for the statical determinateness and stability of trusses in space.

If a point of support of a figure in space cannot move in any direction whatever, we need three components in order to determine the reaction. These components may be conceived as the stresses in bars which connect the point of support with fixed points outside of the figure. In case a point of support is guided on a line, the stresses in two bars are required, and if a point of support is guided on a surface the stress in one bar is required.

For a truss in space let—

a	=	the number of joints in the truss.
n	=	" " bars in the truss itself.
p_3	=	" " fixed points of support.
p_2	=	" " points of support free to move on a line.
p_1	=	" " points of support free to move on a surface.
m	=	" " n of bars in the truss itself plus those bars which act as points of support, as above explained.

We have then the equation—

$$m = n + 3\,p_3 + 2\,p_2 + p_1 = 3a.$$

If $m > 3\,a$ the truss has superfluous bars, and if $m < 3\,a$ the truss can change its shape.

But, although a structure cannot be statically determinate unless the above equation is satisfied, it must not be assumed that the satisfaction of this equation is the only necessary condition.

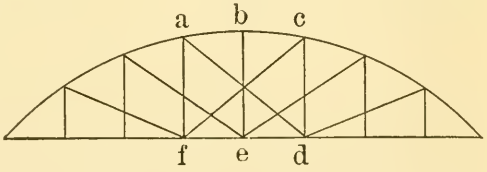
Two conditions are required in order to secure at the same time both the statical determinateness and the rigidity of a structure, and these two conditions may be expressed as follows:

If a truss in space is required to be statically determinate and of invariable form (1st) the equation $m = 3\,a$ must be satisfied, and (2d) the denominator determinant, formed by writing the coefficients of unknowns in a set of equations which express the equilibrium of the truss, must differ from zero.

For a plane figure the relation between the least number of bars and the number of apices is expressed by the equation $m = 2\,a$, and the same condition as to determinants must be fulfilled.

A mathematical proof of these assertions is hardly required, inasmuch as we can always form a judgment about the stability of a truss by computing the stresses and reactions which must be determinate quantities.

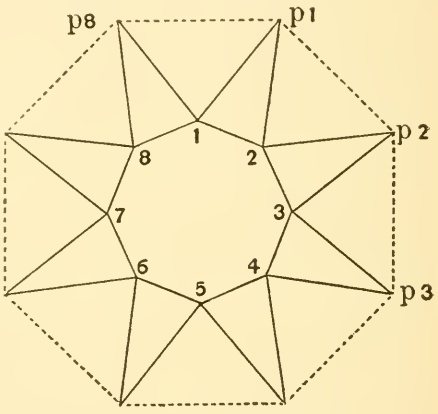
The truss shown in the sketch may serve as an illustration for a plane figure. This truss has one fixed and one loose end, and satisfies the equation $m = 2\,a$. Now, if we simply shorten the middle vertical $b\,e$ until the two top-chord sections $a\,b$, $b\,c$ come into one straight line, parallel to the bottom chord, the structure is at once deprived of its rigidity, and any loading would produce in the hexagon $a\,b\,c\,d\,e\,f$ infinitely great stresses, neglecting the elasticity of the material.



An interesting example of a figure in space which is not rigid, although satisfying the equation $m = 3\,a$, is given by Föppl, and illustrated in the sketch on page 266, $p_1\,p_2 \ldots\ldots p_8$ are fixed points of support of a frame having an even number of sides. It can easily be seen that the apexes 1, 3, 5, 7 can be raised the same amount as the

apices 2, 4, 6, 8 are lowered without the removal of a single bar. The apices are free to turn around the axes $p_1 p_2$, $p_2 p_3$, etc. But if the ring 1, 2, 3,.....8 be given an odd number of sides, the figure is rigid.

The determination of the stresses by statics alone in a rigid truss having only the necessary number of bars does not always admit of a quick solution. On the contrary, it very often involves much labor, unless simplified methods can be discovered. In some cases the problem has been reduced to the computation of plane trusses. For instance, a pyramid with crossed tension diagonals in each panel, and which is open at the top and has no horizontal bracings, is a statically determinate structure. The same may be said if the sides of the structure stand upright, forming a prism. Such a statically determinate pyramid can be conceived as composed of as many plane cantilevers as the structure has sides, the cantilevers being fixed at the bottom. Let us now suppose that an exterior force is acting on any panel point, for instance of an eight-sided pyramid; we can then resolve this force into three components, one of them having the direction of the rafter, and the other two the directions of the two adjacent ring sections. That component whose direction coincides with the center line of the rafter affects only the latter, while each of the other two components affects only that truss in whose plane it acts. From this it follows that the diagonal and ring stresses are independent of the rafter stresses, and that in those rafters on the leeward side which stand opposite to those on the windward side having maximum tension, the wind stresses are equal to zero.



The determination of the stresses in the tower by exact methods is extremely laborious. This is due to its statical indeterminateness which may be reduced, but it can not be entirely eliminated on account of the character of the demand upon the tower.

There is a general method which considers the laws of elasticity in

finding the stresses in a truss having a position in space and any number of superfluous bars, and which is based upon the principle of virtual velocities, a principle which has done such good service in connection with the theory of plane trusses.

An exposition of this principle and of its application to certain trusses can be found in a paper by Professor George F. Swain, M. Am. Soc. C. E., published in the Journal of the Franklin Institute for February, March and April, 1883, and in a paper by Professor William Cain, M. Am. Soc. C. E., "Determination of the Stresses in Elastic Systems by the Method of Least Work," *Transactions* of the American Society of Civil Engineers, 1891.

This method is as follows, assuming frictionless joints:

The general expression of the stresses S in any of the bars can be written as follows:

$$S = U + \alpha_1 X_1 + \alpha_2 X_2 + \alpha_3 X_3 + \ldots\ldots\ldots\ldots\ldots (1)$$

In this equation X_1, X_2, X_3, ... represent the stresses in the superfluous bars, U represents the value of S under the assumption that the exterior forces act on the system of necessary bars alone; that is to say, the superfluous bars are supposed to be removed.

α_1 denotes the stress in any of the necessary bars by forces equal to unity, acting toward each other along the direction of X_1, assuming, as before, that all the superfluous bars are removed.

α_2 denotes the stress in any of the necessary bars by forces equal to unity, acting toward each other along the direction of X_2, assuming again that all the superfluous bars are removed.

The values α_3, α_4, ... are similarly defined.

The values U are independent of the unknowns X_1, X_2, X_3, ... and the values α_1, α_2, α_3..... are independent, both of the exterior forces and of the unknowns X_1, X_2, X_3, ...

The principle of virtual velocities in its application to trusses may be expressed as follows: The algebraic sum of the work of the exterior and interior forces, supposed to be in equilibrium and acting at the joints of a truss, is equal to zero in case of an indefinitely small deformation.

If we consider now the stresses α, and at the same time the virtual displacements, caused by the actual loading, and apply the above principle to this particular case, we arrive at the following equations for the determination of X_1, X_2, X_3.....

$$\Sigma\, \alpha_1 \Delta l = 0;\ \Sigma\, \alpha_2 \Delta l = 0;\ \Sigma\, \alpha_3 \Delta l = 0;\ldots\ldots\ldots\ldots (2)$$

In order to use these equations, we put—

$$\Delta l = \frac{S l}{E a}$$

or, in case temperature stresses are to be considered,

$$\Delta l = \frac{S l}{E a} + e t l \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

In these equations—

l = the length of a bar,

E = the modulus of elasticity,

a = the cross-section of a bar,

e = the coefficient of extension or contraction due to a change in temperature,

t = the uniform change of temperature in one and the same bar.

Substituting now in equation (3) the value of S from equation (1), and in equation (2) the value of Δl thus found, and writing, for brevity's sake—

$$\frac{l}{Ea} = w, \text{ we have—}$$

$$\Sigma\, \alpha_1 (U + \alpha_1 X_1 + \alpha_2 + X_2 + \alpha_3 X_3 + \ldots\ldots) w + \Sigma\, \alpha_1 e t l = 0$$

$$\Sigma\, \alpha_2 (U + \alpha_1 X_1 + \alpha_2 + X_2 + \alpha_3 X_3 + \ldots\ldots) w + \Sigma\, \alpha_2 e t l = 0$$

or—

$$\Sigma\, \alpha_1\, U\, w + X_1 \Sigma\, \alpha_1^2\, w + X_2 \Sigma\, \alpha_1\, \alpha_2\, w + X_3 \Sigma\, \alpha_1\, \alpha_3\, w + \ldots\ldots\ldots + \Sigma\, \alpha_1\, e\, t\, l = 0$$

$$\Sigma\, \alpha_2\, U\, w + X_1 \Sigma\, \alpha_2\, \alpha_1\, w + X_2 \Sigma\, \alpha_2^2\, w + X_3 \Sigma\, \alpha_2\, \alpha_3\, w + \ldots\ldots\ldots + \Sigma\, \alpha_2\, e\, t\, l = 0$$

These sums refer to the external bars, supposed to act at and replace the points of support, as well as to the necessary and superfluous bars.

We have now as many equations as unknowns X_1, X_2, X_3,.....in fact, we can always develop as many equations as we have superfluous bars, and consequently this last set of equations can be solved. After this is done we substitute in equation (1) the values of X_1, X_2, X_3, ... thus found and compute the stresses in any of the necessary bars.

By application of the principle of least work instead of that of virtual velocities we would obtain the same results.

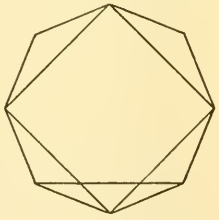
If, now, regarding those bars which take the place of the supports, we suppose them to be subjected to alterations corresponding to the

displacements of the points of support, we can then judge the effect of such displacements on the stresses in the structure.

We know that for certain investigations in connection with the theory of plane trusses it is sometimes of advantage to conceive of a group of bars as a whole, that is as one single plane and rigid figure or disc (German, *scheibe*) without regarding the relative positions of the bars which constitute the group. Professor Müller (Breslau) makes frequent use of such figures; as, for instance, in his investigations of Schwedler domes.

Since it is immaterial, so far as an investigation is concerned, whether the rafters are polygonal or straight, the frame we are considering may be treated in the same way as a Schwedler dome.

It will be remembered that the horizontal braces used in the tower are composed of eight tension members, acting in connection with an octagonal ring. Removing all eight tension members and inserting in their stead five bars (see sketch), capable of resisting compression as well as tension, we produce a rigid figure which performs its work just as well as the original system of bars. Any change in the arrangement of the bars constituting the original system, affects the stresses of these bars only, leaving the stresses in all the other bars unaltered, provided the original form of the octagon remains the same, for the virtual displacements of the apices are not affected by the means which we choose to effect the rigidity of the ring. These considerations enable us to calculate the stresses in the tower-cap. The forces acting on the tower-cap are known, if we substitute a brace, as shown in the sketch, and compute all the stresses. If now we pass a section through the tower-cap at any point, the determination of the stresses will be the same as that of a curved beam fixed at both ends.



The effect of improper fitting on the stresses may be determined by employing the original equations and by assuming for those bars whose lengths are not exact a proper temperature which is added to or subtracted from or substituted for the original values of t. The alteration in the length of a bar whose temperature is raised t_1 degrees is $e\,t_1\,l$, and consequently, if a bar is too short by a quantity $= c\,l$, its temperature must be raised $\frac{c}{e}$ and the temperature of that bar, which is too

long by $c\,l$, must be lowered $\frac{c}{e}$. Assuming now the original temperatures of the bars after their insertion, we must attribute to those bars whose temperature has been raised, a change of $(-t_f)$ degrees, and to those whose temperature has been lowered, a change of $(+t_f)$.

Where a thorough knowledge of the conditions of a structure is required, all secondary stresses must be included in the investigation. Exact methods for finding secondary stresses, even in plane trusses are, comparatively speaking, of recent development; while with regard to trusses in space this subject, so far as the writer knows, has hardly been touched upon by any author.

Such methods have the disadvantage that they require an estimate of the sections which are the object sought. It is therefore necessary to use some kind of an approximate method to determine the sections. These sections may then have to be changed, in order to comply with the results of the investigation, and if the changes thus required are of considerable extent, all the calculations may have to be repeated.

If we consider that the calculation of the secondary stresses due to yielding of the ground and to deformation of the masonry presents insurmountable difficulties, if we further consider that the modulus of elasticity in any given material is a quantity of uncertain amount, and that even our usual assumption as to the distribution of the dead weight is in many cases only approximately true, we see at once that the true stresses can never be computed, and that we must content ourselves with working stresses of lower intensities than would be permissible if the stresses could be accurately known. The writer would never advocate the adoption of trusses with superfluous bars, except in cases where they can be shown to have some marked advantage over others.

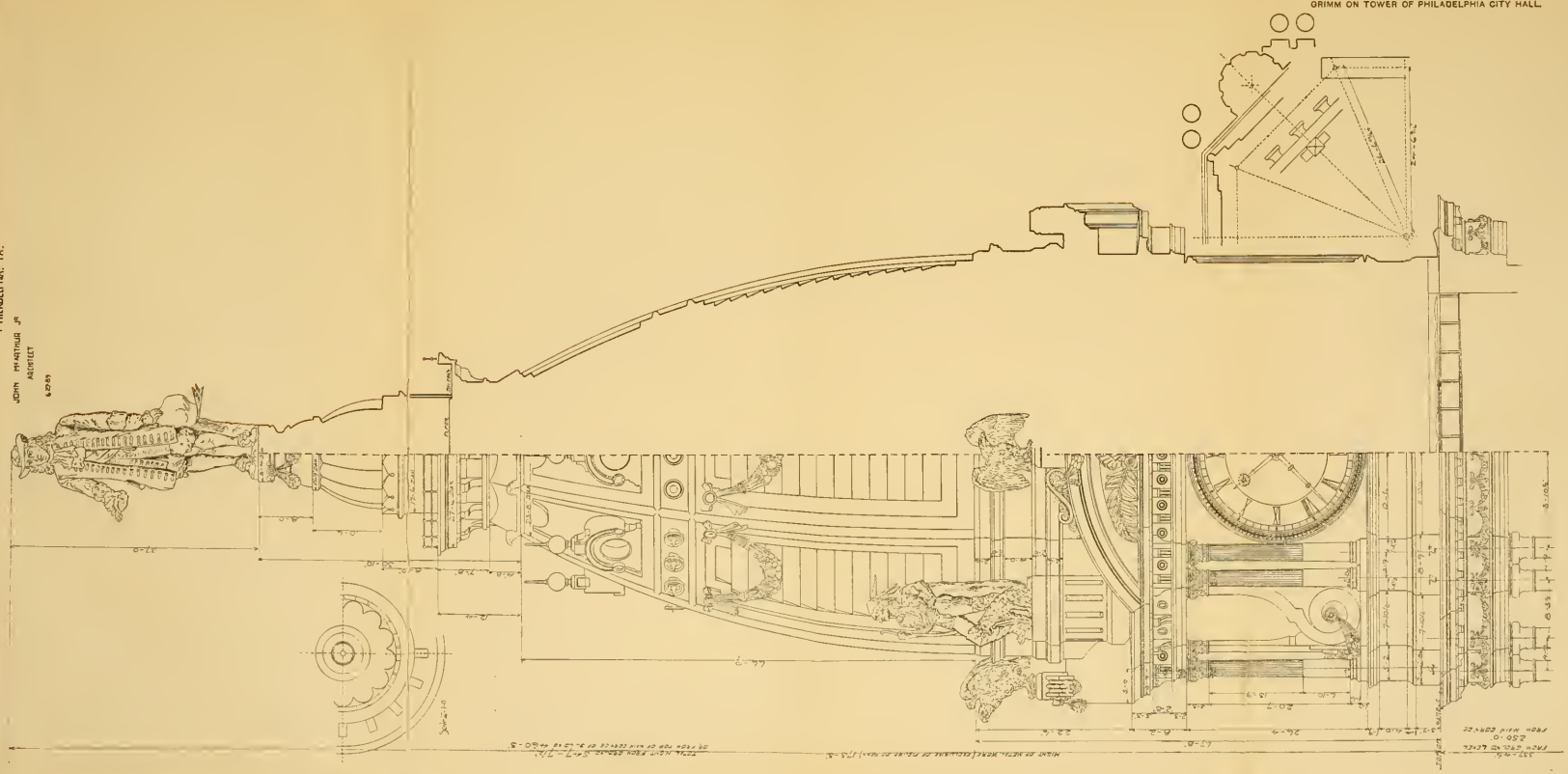
Since instruments are at our disposal by means of which we can ascertain alterations of less than $\frac{1}{10\,000}$ of 1 in. in the lengths of the bars, the surest way to find the true stresses would be to measure these alterations. From these the stresses can easily be computed.

The writer could not obtain an account of the cost of the tower, but to judge from general information he thinks the cost, inclusive of erection, will be from $650 000 to $700 000.

Believing that it will interest the members of this Society, the writer will give, in conclusion, a table showing, in the main, the hori-

NEW CITY HALL
PHILADELPHIA, PA.

JOHN MCARTHUR JR
ARCHITECT



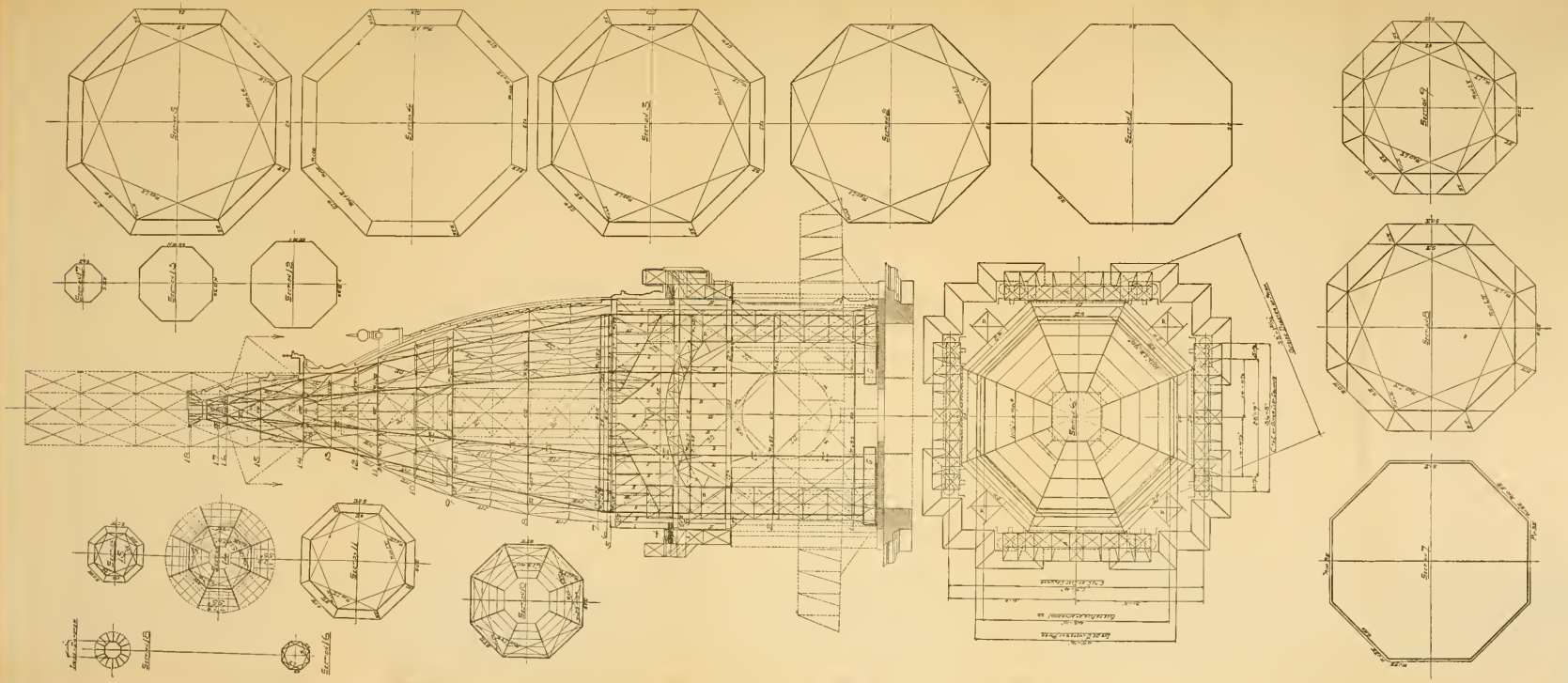
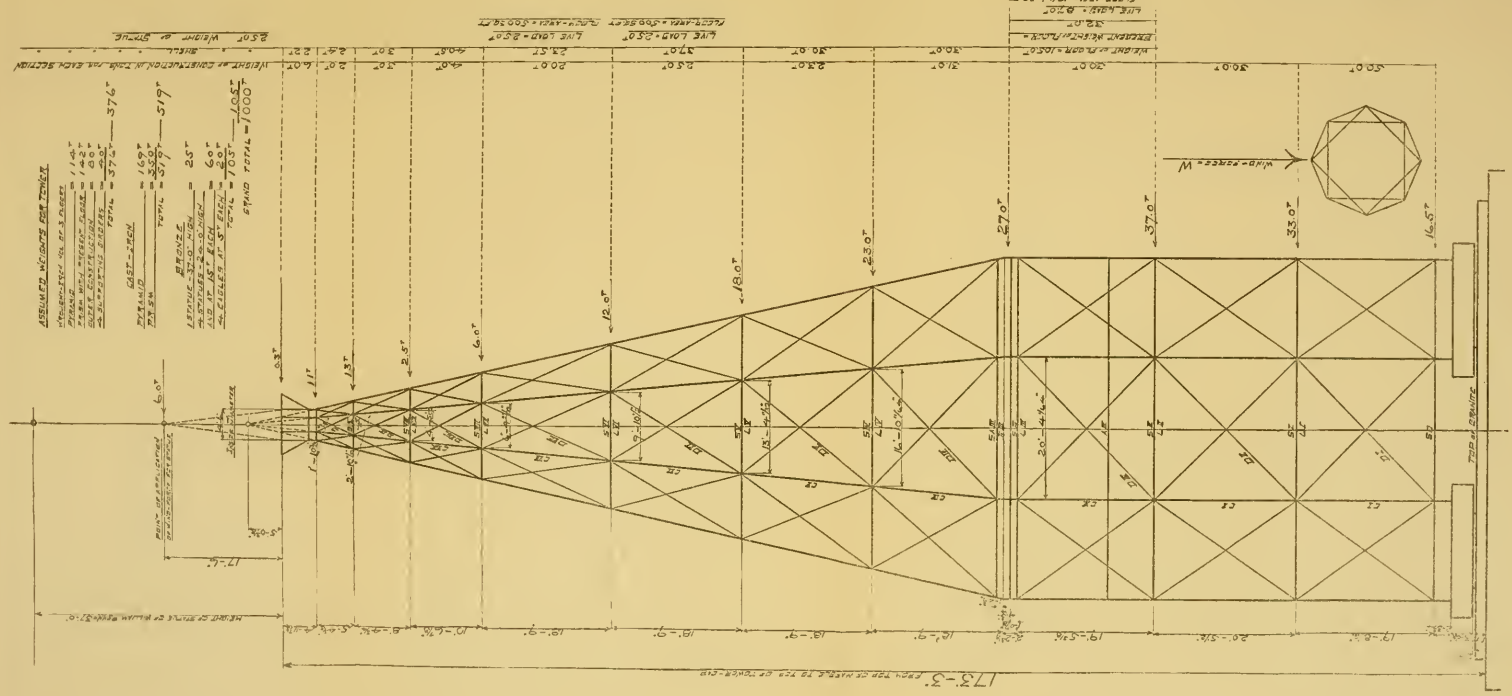
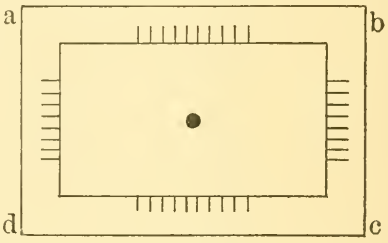


PLATE XXXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 694.
GRIMM ON TOWER OF PHILADELPHIA CITY HALL.



zontal deflections of the masonry tower under the influence of the sun. The readings, with respect to the deflections and temperatures were, as a rule, taken three times a day, excepting Sundays, and were commenced on the 1st of March, 1892. The table covers a period of one year from that date to the 1st of March, 1893. The maximum, minimum and average movements, north, south, east and west, as also the number of movements, are given for each month. The table shows also the maximum, minimum and average temperatures, as also the number of temperature observations, for each month. The temperatures were taken inside the tower at a point 230 ft. below the top of the stone work. The deflections were measured by means of a plumb line 230 ft. long, consisting of a heavy weight and a thin piano wire, suspended from a point at the center of the top of the stone tower. The plumb line at its lower end passes through a small, rectangular, wooden frame *a b c d*. This frame rests on a wooden box, containing molasses, into which the weight of the plumb line is immersed. Each side of the frame is divided in the neighborhood of its center into thirty-seconds of an inch. By the use of a steel straight-edge the deviations were easily ascertained.



Since the stone tower is 337 ft. 4½ ins. high, we should add about 47% to the deviations, in order to obtain those for the total height.

Movements of the Masonry Tower in Inches.

	North.			South.			East.			West.		
	Max.	Min.	Average.	Max.	Min.	Average	Max.	Min.	Average.	Max.	Min.	Average.
1892.												
March	3/16	1/32	5/64	3/16	1/32	5/64	1/8	1/32	3/64	1/8	1/32	3/64
April	9/32	1/32	3/32	1/16	1/16	1/16	3/32	1/16	1/16	1/16	1/32	3/64
May	7/32	1/16	11/64	0	0	0	3/32	3/64	1/16	3/64	1/32	3/64
June	9/32	1/16	11/64	0	0	0	3/32	3/64	1/16	3/64	1/32	3/64
July	3/8	1/16	5/32	0	0	0	1/4	1/32	1/16	1/32	1/32	1/32
August	1/4	1/32	3/16	0	0	0	3/32	1/32	3/64	3/32	1/32	1/16
September	5/16	1/16	5/32	0	0	0	1/8	1/32	3/64	1/32	1/32	1/32
October	1/4	1/32	1/8	0	0	0	1/8	1/32	1/16	0	0	0
November	1/4	1/32	1/16	3/32	1/32	3/64	3/32	1/32	1/16	1/32	1/32	3/64
December	5/64	1/32	5/64	1/8	1/8	1/8	3/32	1/32	3/64	1/4	1/32	5/64
1893.												
January	1/8	1/32	3/32	5/32	1/32	5/64	1/16	1/32	1/32	3/16	1/32	3/64
February	5/32	1/32	5/64	1/8	1/32	3/64	1/16	1/32	1/32	1/16	1/32	1/32

Number of the observations of movements.				Temperatures.			Number of the observations of the temperatures.
North.	South.	East.	West.	Max.	Min.	Average.	
29	40	24	24	58	24	40.5	65
68	1	41	9	78	39	54	78
72	0	30	7	82	52	65.5	78
76	0	64	2	90	64	78	76
73	0	46	7	98	64	80	73
76	0	61	3	93	70	79	77
76	0	54	1	88	58	70.8	76
74	0	56	0	78	44	60.5	78
55	14	66	2	67	29	47.6	78
62	4	19	11	64	16	35.8	81
5	52	6	37	44	2	26	78
32	19	12	17	54	14	36.4	72

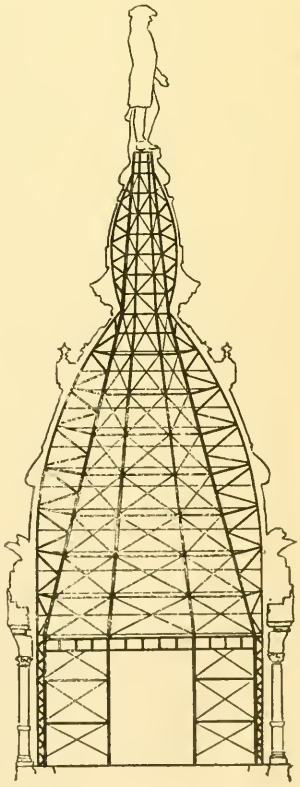
DISCUSSION.

JOSEPH M. WILSON, M. Am. Soc. C. E. (by letter).—At the request of Mr. John McArthur, architect, City Hall, Philadelphia, I did a considerable amount of expert work for him in 1880 to 1883 on the proposed tower for the City Hall.

I had the weights and pressure per square foot computed for the several horizontal sections of the tower, which I reported to him in April, 1880, with recommendations, and it was on my recommendation that he reduced the weight by changing the upper portion of the work, as originally designed, to iron, and also making some changes in the thickness of the walls below.

I also made a design for a skeleton iron construction for the upper part of the tower and furnished him with general drawings for this work, which were sent to him on May 7th, 1880.

The design was adopted by Mr. McArthur and placed on his drawings. The framework of this design was of entirely open construction, consisting, for the lower portion, of an octagonal prism, and, for the upper, an octagonal pyramid, the inner or heavy skeleton being the pri-



mary, and the outer or light one intended to support the shell, both being braced together. The entire interior is open, with a floor on girder construction at the junction between the octagonal prism and the pyramid.

Not having seen Mr. Grimm's drawings I do not know how the two designs compare, but I submit this as perhaps an interesting item in the history of the work on this tower.

We (Wilson Brothers & Co.) designed the crane and accompanying machinery used in the erection of the stone work of this tower, and also the high false works used at the central entrances of the building during construction.

I have designed and had constructed several roofs covering spaces nearly square in shape, 80 to 90 ft. square, in which the thrust was taken by ties around the exterior walls, there being no ties crossing the space covered. Among these I might mention the central part of Machinery Hall, Centennial Exhibition, where nave and transepts crossed, and the main waiting-room of the former passenger station of the Pennsylvania Railroad at Jersey City. The main rafters were placed on the diagonals of the square, meeting at the apex of the roof, and the construction formed what might be called a square dome, the whole space being left open up to the roof.

George Hill, Assoc. M. Am. Soc. C. E.—The designing and erection of towers is so common an occurrence, that the interest therein is less in the principles than in the application thereof. In the present instance, it would seem as though certain precautions had been neglected. If I correctly understood Mr. Grimm, the sizes of the members for the preservation of the form of the tower in plan were determined arbitrarily, which in so important a construction, and especially one of such size in plan, would seem to be a wholly unnecessary compliance with an unreasonable architectural requirement.

The construction of the tower in two parts, one intended to resist the wind and other loads, and the other simply to carry the covering of the tower, is such as to inevitably give rise to different amounts of expansion between the two, when subjected to the sun's heat, the outer shell, which is directly bolted to its supports, being warmed to a much higher temperature than the inner or main portion. If the tower is 175 ft. high, the approximate amount of expansion would be $1\frac{3}{4}$ ins., and if we should assume that the outer portion expanded twice as much as the inner one, which would seem to be reasonable, there would be a difference between the two of $\frac{7}{8}$ in., which would have to be accommodated either by springing the connecting brackets, or else by wrinkling and buckling the thin cast-iron shell; and this would, inevitably, lead to very annoying leaks if to nothing more. In my own experience, I have known of rafters 20 ft. long in a mansard roof, covered with fire-proofing and then with copper, to expand and contract

under the influence of the sun, coming on for a few hours near noon, and then passing off a sufficiently great amount to crack the plaster where the mansard joined the side walls, indicating a movement of $\frac{1}{16}$ in. It would seem as though the same care which dictated the plating of the tower with aluminum would have provided for the free motion of the covering.

A final point would be to question the advisability of using any untried material for the making of a joint under such trying conditions. Long experience with rubber as an insulating medium on electric lighting wires has shown the great difficulty of obtaining anything which is permanent in its character, which would at once raise the question as to the life of any cement with rubber as an essential part. In addition to this, bitter personal experience has shown me that in important constructions it is not enough that a material should apparently contain all of the elements that are desirable, but we should go further and have proof of its satisfactory operation under analogous conditions.

Mr. J. A. McDonald, M. Am. Soc. C. E.—Nothing has been said in this paper, so far as I am aware, about the factor of safety allowed for wind pressure. It is stated that the stresses have been calculated for 50 lbs. to the square foot. It is very important that we should consider the question of the greatest allowable stresses on such structures. You might get the 50 lbs. per square foot three times in a year, or, perhaps, once in a lifetime, and, therefore, towers exposed to wind pressure only should be treated differently to structures subject to other loading, and not have the same factors of safety. The stresses do not alternate rapidly, and the maximum stresses are so seldom, if ever, attained, that it appears to me we might apportion our material very closely up to the elastic limit. The question, I think, for engineers to decide is, how close it would be safe to go towards the elastic limit of the material with a structure which is stressed chiefly by wind pressure.

The paper does not give how closely the calculated stresses come to the elastic limit of the material. I think, as I said before, that in these structures we might apportion our material to come very close indeed, to the elastic limit of the material itself.

J. Foster Flagg, M. Am. Soc. C. E.—I would like to ask one question. I understand that the shell of the dome is first coated with copper and then with aluminum. Is there any difficulty in depositing aluminum directly on iron, or for what other reason is copper used?

Was it necessary to first coat the surface of the iron with copper in order to secure a firm attachment of the aluminum?

C. R. Grimm, M. Am. Soc. C. E. (by letter).—Mr. Wilson's statements certainly form some interesting items in the history of the work upon this tower, and I only wish to add that I had no knowledge, in

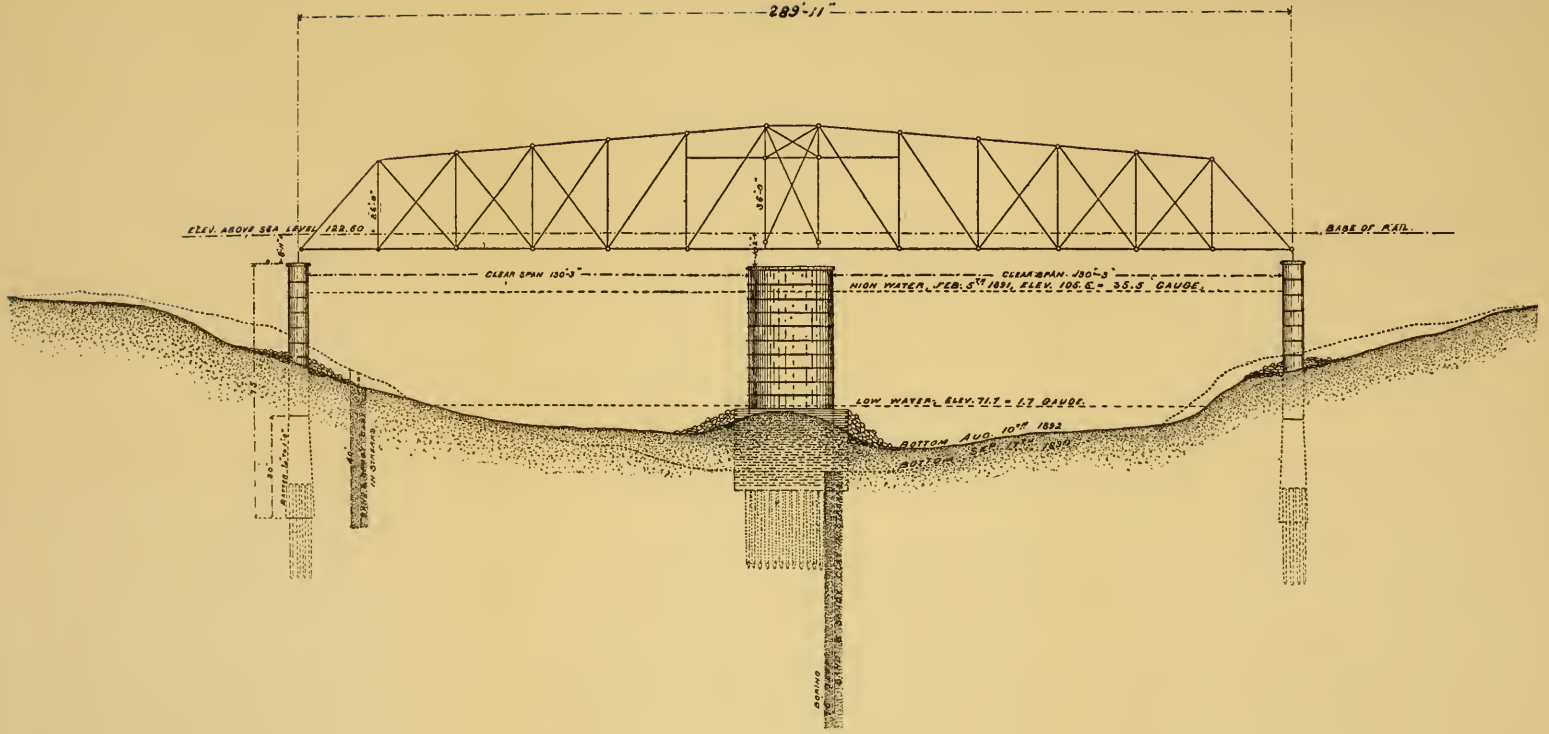
any shape or manner, of his design of an iron construction for the upper part of the tower until the day when I read my paper.

Mr. Hill must have undoubtedly misunderstood me when he thinks that an unreasonable architectural requirement interfered with the determination of the sizes of the members for the preservation of the form of the tower in plan. This subject is fully explained in the paper. Concerning his remarks with respect to the shell, I will confine myself to the remark—since the design of this shell has been outside of my province—that the bottom sections are not bolted to the top of the stone tower, and that the louvres are independent of the shell.

I regret very much that I am not in a position to satisfactorily answer Mr. McDonald, who points out that the paper does not give how closely the calculated stresses come to the elastic limit of the material. Without regard to a few specimens of square rods which were tested, no tests have been made in order to obtain knowledge of the qualities of the material, for reasons entirely unknown to me. On the other hand, the manufacturer was requested to furnish a material in compliance with the standard specification of bridge-builders.

In answer to Mr. Flagg's question, I must state that it is necessary to first coat the surface of the cast iron with copper, because if aluminum is deposited on any kind of iron, such an extremely thin and porous coat is produced that it would never answer the purpose for which it is put on in the present case.

PLATE XXXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 695.
KELLEY ON RECONSTRUCTION OF PIVOT PIER.



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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

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695.

(Vol. XXXI.—March, 1894.)

THE REMOVAL OF A DEFECTIVE PIVOT PIER, AND ITS RECONSTRUCTION.

By HOWARD G. KELLEY, M. Am. Soc. C. E.

READ DECEMBER 6TH, 1893.

On the afternoon of Saturday, September 13th, 1890, the St. Louis, Arkansas and Texas Railway draw span, with four panels of trestle approach, crossing the Ouachita River, at Camden, Ark., was wrecked, a 60-ton Mogul engine and nine loaded freight cars falling into the river with the collapsed structure. Fortunately, no one was injured.

The span destroyed was a combination Howe truss draw of 290 ft., total span, with curved iron top chord and iron bottom chord, erected in 1883, the road at that time being narrow gauge; all the wooden web members and the floor timbers had been renewed two years previous to the accident. The span rested upon a pivot pier composed of a cluster of cylinders; the central cylinder was 6 ft. in diameter, surrounded by six cylinders, each 4 ft. in diameter, all of the cylinders being braced together. The end rest piers were ordinary framed bridge seats on foundation piling.

By referring to Plate XXXV, it will be seen that the bed of the river was about 70 ft. below grade in its deepest portion, the bottom being

composed of sand, with 4 to 6-in. strata of sandy clay at intervals of about 10 ft. in depth.

Prompt measures were taken to remedy the disaster, and the next morning a force of company men was at work on the wreck. The broken span was pulled clear of the pivot pier down stream, the engine and cars rolled out of the way to make room for the erection of a temporary trestle, while another force was cutting from timber land a few miles distant the necessary long pine piling for this structure, the length of piling cut running from 70 to 85 ft. By night one track-driver commenced driving from the south end, and by the following afternoon a second track-driver commenced working from the north end, the latter driver having been brought from the Texas division and run over foreign lines to reach the work from the north side.

Five piles to the bent were driven, braced with 3 x 10-in. plank, the bents being spaced 14 ft. centers, cribbing being used over the pivot pier. The work was continued night and day, and by noon of Friday, the 19th, the trestle was completed and trains crossed the bridge, the regular train service of the road being continued without interruption until the final rebuilding of the new bridge.

The bridge having been originally erected under an act of Congress, specifying the length of clear span, application was made to the War Department for the privilege of reducing the length of span. Pending this decision the wrecking of the old span and rolling stock from the river was carried on, but it was not until the latter end of November that notice was received denying the application. A contract was immediately let to the Detroit Bridge and Iron Works for the superstructure, which was to be 290 ft. between centers of end pins, designed in conformity with Theo. Cooper's Specifications for Class A. The design was made by Mr. J. W. Schaub, M. Am. Soc. C. E., the engineer of the Detroit company.

An examination of the pivot pier showed extremely poor concrete, much of it being uncemented. The penetration of the cylinders in the sandy bed of the river varied from 14 to 16 ft., no piling having been driven in the bottom of them during the original founding. The pier was, therefore, condemned, and plans for its reconstruction considered.

To alter the location on the existing alignment of the bridge would have required a heavy increase in the length of the draw span, while

to have changed the alignment of the road to make a different crossing, up or down stream, would have required an expenditure of money which made it useless to take the plan into consideration. It now became apparent that the only feasible plan was the removal of the existing cylinder cluster and replacing it with a cylindrical pier.

The Ouachita is a turbulent river, subject to frequent extreme floods. The United States records show rises of 15 ft. in one night, but in the vicinity of Camden it is not subject to heavy scour or change of channel. It carries heavy drift in flood waters, with many large logs, and has sufficient current to cause logs to dive when striking an obstruction, making any plan for submerged pile foundations dangerous.

The contraction of the water-way, due to the trestle work and the wreckage, had produced some scour, making the depth in the middle of the river about 20 ft. below low water; therefore, the use of a pneumatic caisson, through which the old pier would necessarily be removed, was considered impracticable from both a constructive and financial point of view; but by the use of an open caisson, or coffer-dam, sunk to a sufficient depth below the bed of the river, the defective pier might be removed economically and a new one constructed within the walls of the caisson. The difficulty of handling such a caisson, and the gravity of the work under the surrounding conditions, were fully appreciated, but it was judged to be the only feasible method, and the plan was developed as follows:

"General Plan of the Work.

"*First.*—Sink a double-walled caisson or coffer-dam to a penetration of 10 ft. below the scoured bed of the river, the caisson to surround the cylinder cluster.

"*Second.*—Fill the interior space in the coffer-dam with piling, driven to as great a depth below the cutting edge as possible.

"*Third.*—Seal the interior with concrete, pump it out, cut off the piling and the six small cylinders below low water, capping them all with square timber, upon which a grillage of square timber should rest, said timber to surround and be fitted to the central cylinder, which, as stated before, was 6 ft. in diameter.

"*Fourth.*—Cut off the central cylinder about 2 ft. from the top of the coffer-dam.

"*Fifth.*—Upon the grillage erect a cylinder of riveted steel plates 24 ft. in diameter and 44 ft. high, filled with concrete, to be surmounted with a limestone coping 24 ins. thick."

Estimates were made upon this general outline of plan, and the plan submitted to contractors for bids, with the privilege of their submitting bids based upon plans of their own, if they so desired.

The reputation of the Ouachita River as being a treacherous and turbulent stream was well known among substructure contractors; therefore no surprise was felt when they declined to bid upon plans of their own. But when the lowest bid based upon the plan outlined above was received, and found to exceed by 40% the estimate made for the actual cost, some explanation to the management of the Company appeared to be in order.

The subject was then carefully considered by the General Manager, Mr. W. B. Doddridge, and the writer, feeling confident that the work could be done in the manner and for the amount of money he had specified, was authorized to proceed with the work with the regular Company forces. The work was completed precisely in accordance with the prepared plans and specifications, including the cleaning up and shipping to headquarters of all machinery and surplus material, at a total cost of 85% of the original estimate, and $60\frac{7}{10}\%$ of the lowest bid received.

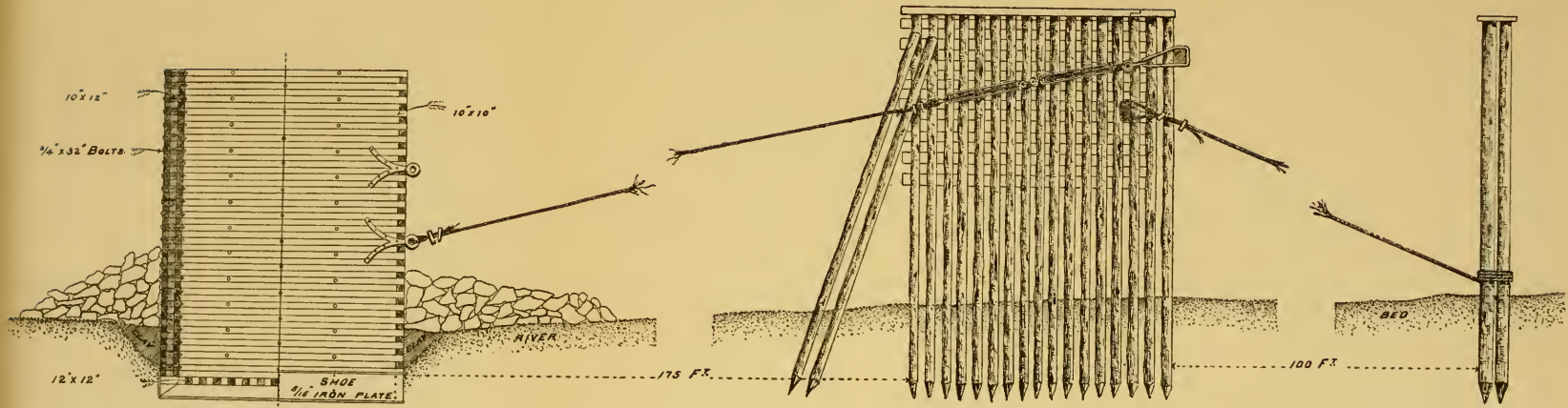
History and Description of the Work.

Active work was commenced on December 18th, cleaning up all remaining portions of the wrecked bridge, and, by means of a submarine diver, fastening chains to the engine, which had not up to that time been secured. A submarine diver was employed by the month, the Company furnishing the diving outfit. Especial care was taken to remove every obstruction, such as drift and old logs, from around the pivot pier within the area of the proposed coffer-dam, the diver using a strong water-jet to assist him in this work.

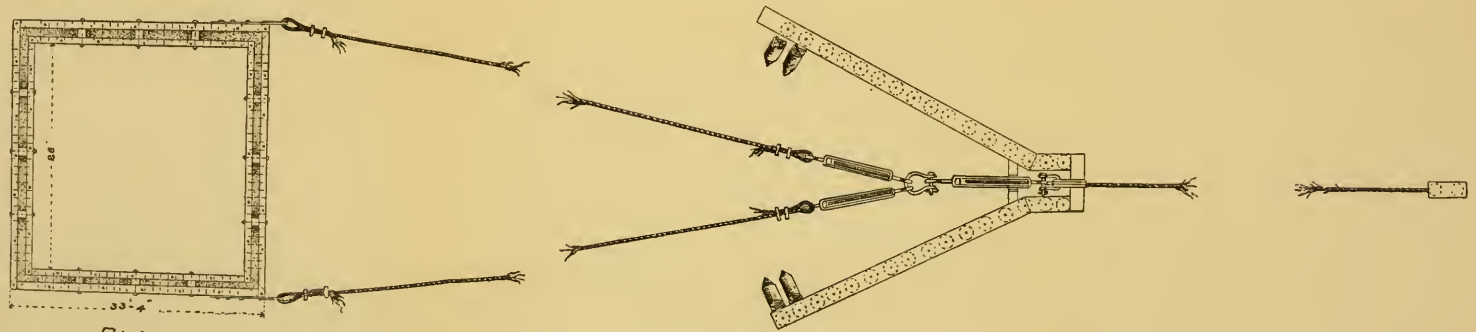
To protect the coffer-dam from the pressure of the current, a breakwater was driven about 175 ft. up stream in V shape, with the wings projecting down stream and flaring out slightly beyond the sides of the coffer-dam (see Plate XXXVI). Each wing consisted of a line of fourteen pine piles braced with two battered piles at its down-stream end, to better resist fields of drift; the wings were capped with a 12 x 12-in.

PLATE XXXVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 695.
KELLEY ON RECONSTRUCTION OF PIVOT PIER.

COFFERDAM AND BREAKWATER



HALF SECTION HALF ELEVATION



PLAN

timber 36 ft. long, and slatted up on the up-stream side with rows of 2 x 8-in. plank. The nose of the breakwater consisted of four heavy pine piles capped with 12 x 12-in. timber, on which a platform was built.

About 100 ft. up stream from the nose of the breakwater two pine piles were driven and chained together ; a wire rope made into a sling at one end was passed around these piles, the sling allowing the line to settle to the bottom ; the other end was then carried to the nose of the breakwater and a strain put upon the line, which was then made fast, thus acting as a guy to stiffen the breakwater.

To assist in the river work, two decked barges had been built, dimensions 12 ft. x 32 ft. x 2 ft. deep, being made very strong and stiff; also, two open barges, 8 ft. x 16 ft. x 18 ins. deep ; also, some floating runways, which could be run from the shore to any part of the work desired.

It was presumed that the coffer-dam could be held in position laterally by bracing against the pivot pier when necessary, but to support it against the current wire cables 1¼ ins. in diameter were led from the different anchor-straps on the sides of the nose of the breakwater, as shown in Plate XXXVI, additional anchor-straps being added when necessary. By the aid of adjusting screws and a crab winch on the nose of the breakwater, sufficient strain could be put on the cables to hold the coffer-dam in position up and down stream and twist it as occasion might require. This plan worked admirably throughout the sinking.

Some additional work was also necessary on the temporary trestle to make room for the coffer-dam, and a clear channel span of 24 ft. was required to permit rafts to pass through, the track being carried by some trussed stringers.

The trestle was also stiffened to resist floods and accumulations of drift, by running six wire cables 1¼ ins. in diameter each from it up stream to trees on the bank, where the cables were made fast.

On February 14th, everything having been prepared, and the engine and wreckage taken out of the river, the iron shoes forming the cutting edge of the coffer-dam were floated out to the pivot pier on the barges, the shoe being in four sections; it was assembled in place around the pier and riveted together, and the first four courses of wall built upon it. Its weight was supported by the barges and eight crab winches stationed on the platform built in the trestle overhead.

The shoe was then filled with concrete, and on February 18th it was lifted clear of the barges by means of the crab winches, then lowered into the water until all but the top course of timbers was submerged; on this date the gauge read 20.3 ft. above low water, which gave a total depth at the pivot pier of about 39 ft.

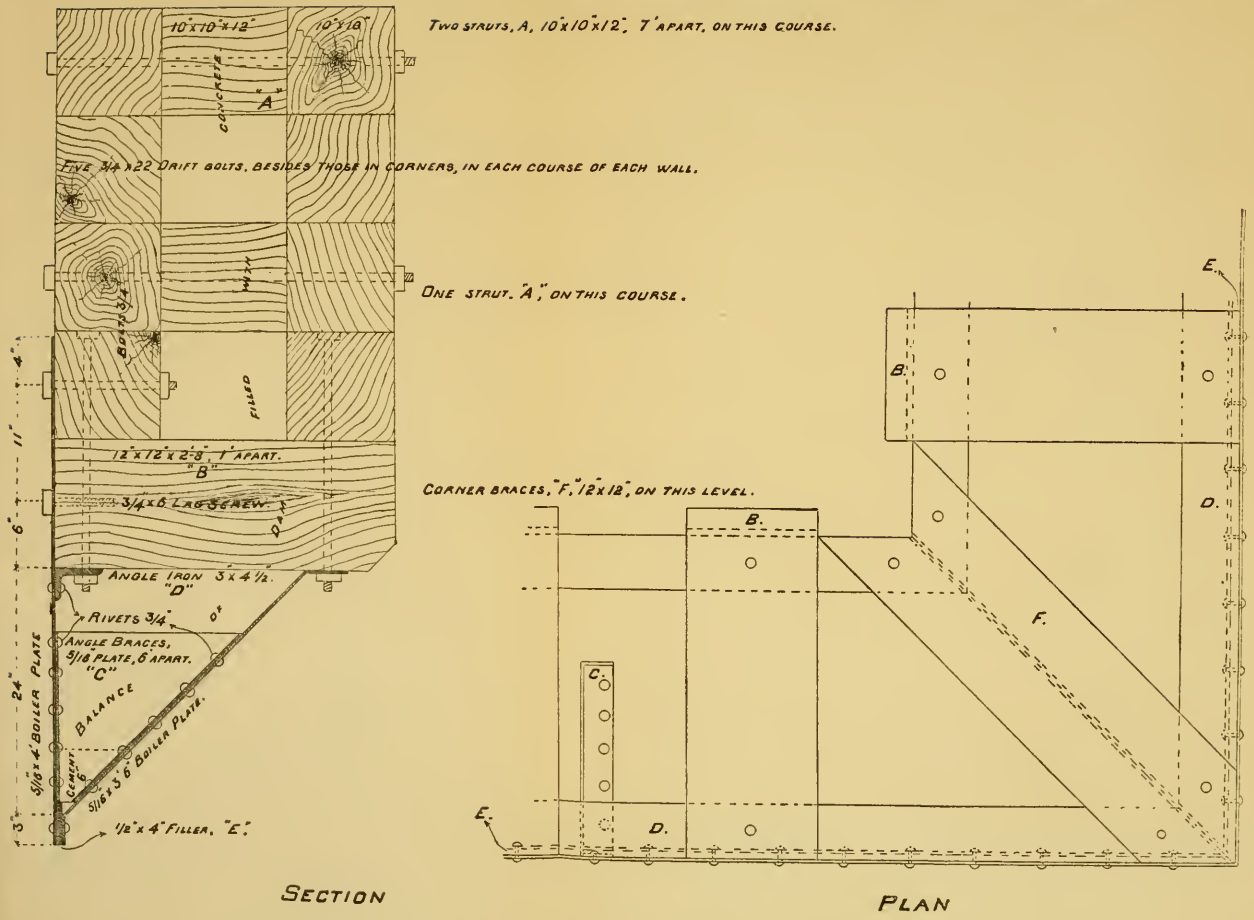
By referring to Plates XXXVI and XXXVII, the construction of the coffer-dam can be seen. In plan it was 28 ft. square in the clear inside, and 33 ft. 4 ins. outside; the double walls were of 10 x 10-in. timbers, ordered 32 ft. and 34 ft. long, so that no splicing would be necessary in the walls. Each course of timbers was drift-bolted to the course below with five round iron drift-bolts, $\frac{3}{4}$ x 22 ins. each side, besides the bolts in the corners. The coursing timbers were not halved or framed together, but were simply cut square and fitted in place with alternate butt and lap joints.

At every alternate course strut blocks were placed between the double walls; these blocks were of 10 x 10-in. timber, 12 ins. long, which was the clear space of the double wall. A $\frac{3}{4}$-in. hole was then bored through both walls and the block and the timber drawn together by an iron bolt $\frac{3}{4}$ x 32 ins. long. This system was carried around all four sides of the coffer-dam and continued to the top course.

The iron shoe was of $\frac{5}{16}$-in. plate iron, made in a V-shaped trough section with the line of the outside plate vertical, and in line with the outside wall of the coffer-dam (see Plate XXXVII); they were stiffened and riveted together with plates and angles. On top of this shoe rested, first, a series of foundation blocks 12 x 12 ins. x 2 ft. 8 ins., placed 12 ins. apart in the clear, and mitered at the corners of the coffer-dam, each block being bolted to the shoe as shown. On these blocks rested the double walls, the first course being bolted down with the same bolts by which the foundation blocks were bolted to the shoe.

The shoe was then filled with concrete up to the top of the foundation blocks. Tackle had already been fastened to all four corners of the coffer-dam and to the centers of each side wall. This tackle was suspended from the ends of trussed beams placed across the top of the track circle of the old pivot pier, the truss rods leading up and over the stringers of the temporary trestle; the hauling lines led through snatch blocks to the eight crab winches before mentioned. Everything being in readiness, the winches were manned and a steady strain put upon all eight lines, lifting the constructed portion of the coffer-dam

DETAILS OF SHOE



up slowly from the barges, which were then hauled out of the way. The coffer-dam was then lowered until only the top course of timbers remained out of the water. The total estimated weight thus lifted was 99 200 lbs.

The construction was carried on by bolting the wall timbers in place and lowering away on the tackle until the buoyancy of the timbers somewhat relieved the tension, when more concrete was added in the space between the walls until the weight was sufficient to keep a constant strain on the tackle, experience confirming the original opinion that it would be more manageable when heavy than when partially buoyant.

This concrete was mixed on one of the small barges alongside and was deposited in place through a box chute 10 ins. square, the leakage of water through the double walls when first made being sufficient to keep the 12-in. wall space nearly full of water.

Meanwhile a force of men was employed preparing for the later portions of the work; the steel cylinder material had been ordered, and was being constructed and inspected at the shops; concrete rock was being received by train and unloaded at a convenient place on the river bank; the platform and shed for mixing concrete were erected, and all necessary provision and details for the economical and successful prosecution of the work were given due attention. One of the most successful features of the work was that no portion of it was ever held or delayed for want of necessary material or preparation beforehand.

On March 4th the coffer-dam landed on the bottom of the river, 44 courses of timber wall having been built on top of the shoe, giving a total height of 40 ft. 8 ins., the gauge reading 20.3 ft. The wall space was then filled to the top with concrete. The coffer-dam was then carefully set in position by means of the guy lines. A transit was used to determine the true position, and to set reference points on the temporary trestle, by which the position of the top of the wall could be determined at any time.

The plan designed for the sinking was for a diver to use a strong jet of water delivered through a nozzle in a 2-in. rubber steam hose. The diver worked inside the coffer-dam, jetting the material away from the cutting edge and toward the four corners of the coffer-dam. By reference to Plate XXXIX it will be seen that vacant spaces existed at these points between the walls of the coffer-dam and the small cylinders, these spaces being sufficient to permit the use of a Hayward dredge.

A system of four derricks had been erected on the trestle work overhead, as shown in Plate XXXVIII, and a hoisting engine set up, by which the dredge could be handled in all parts of the coffer-dam and could be passed from one derrick to another over the coffer-dam and beneath the decking of the trestle, without landing it.

There was but little rip-rap found around the pier, and this was removed by means of the dredge, the larger pieces being loaded into the dredge by hand by the diver.

The weight of the coffer-dam now enabled it to be handled in a manner somewhat similar to a pneumatic caisson, for by means of the jet the cutting edge could be thrown up or down stream, or across, as occasion demanded, while at the same time the guy lines held it in whatever position was required above.

The timber walls were continued up as the coffer-dam penetrated the bottom, until it reached a total height of 48 ft., or 52 ft. above the cutting edge. It was not deemed expedient to build it any higher, but the fluctuating floods were allowed to submerge it, this submersion occurring twice during the sinking without any harm resulting other than the delay to the work, while the coffer-dam was submerged.

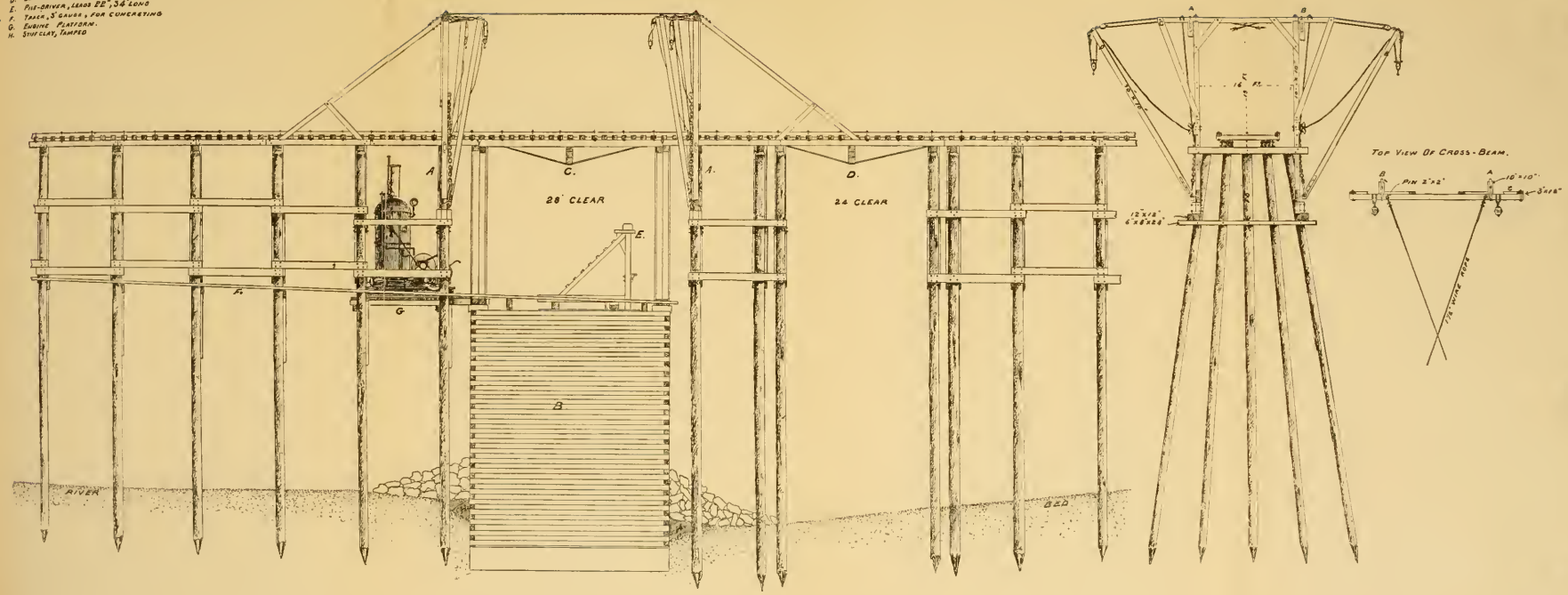
The following is a condensed table of gauge records, showing the highest and lowest points of fluctuation during each month, with the dates :

GAUGE RECORD.

DATE.	ELEVATION ABOVE LOW WATER.	
	Highest.	Lowest.
December 20, 1890		6.2
" 29, "	19.0	
January 1, 1891		14.3
" 14, "	31.3	
" 26, "		12.3
February 5, "	35.4	
" 16, "		19.1
" 26, "	32.2	
March 6, "		17.1
" 12, "	32.0	
" 31, "		19.1
April 3, "	21.9	
" 16, "		12.3
" 26, "	29.8	
May 14, "		7.1
" 21, "	16.0	
" 31, "		7.5
June 9, "		4.6
" 20, "	9.4	
July 1, "		5.0
" 8, "	9 0	
" 27, "		3.8

PLATE XXXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 695.
KELLEY ON RECONSTRUCTION OF PIVOT PIER.

TRESTLE AND DERRICKS



This table shows a record of four successive floods higher than 30 ft. above low water, making a total head of water to work against of over 50 ft. At a gauge of 30 ft. the water would overflow the high south bank where the work and material yard was located, compelling the boarding outfit to pull out to higher ground at Camden station, about one mile away, and stopping all work except such as could be done from the trestle or the barges in the river.

During the sinking a ditch about 4 ft. deep formed around the outside of the walls of the coffer-dam. This was expected, but on the upstream right-hand corner some scour commenced, cutting a trench through the coffer-dam diagonally across to the middle of the downstream wall. This might have proved serious, but was successfully stopped with sandbags deposited in the ditch outside, and also by keeping that corner of the coffer-dam hard down by extra weighting it with sandbags laid on top of the wall, the broad walls of the coffer-dam affording an excellent gangway for the men to work upon.

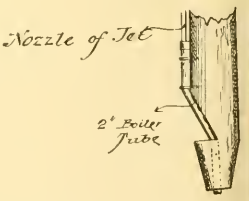
On April 4th, the cutting edge had reached an average penetration of 10 ft. below the river-bed, having been landed within 2 ins. of parallel with the axis of the bridge, the center being 1½ ins. down stream and 1 in. further north than the mathematically true position. Gunny sacks, partly filled with concrete, were then placed by the diver inside and underneath the overhang of the shoe, filling the space entirely, as shown in Plate XXXIX. A row of sacks were then laid in the ditch outside and against the walls, after which the ditch was filled with stiff clay, on top of which rip-rap was placed. As the clay was deposited, it was tamped in a simple and satisfactory manner by having the diver walk around the dam, forcing the clay hard into place with his iron shoes. The diver being an old experienced man, it was not necessary to pump sufficient air into the dress to make him buoyant when on the bottom while the protection afforded by the breakwater prevented the river current sweeping him off his feet.

The cribbing was now removed from over the old pivot pier; framed bents were erected on the walls of the coffer-dam, and some trussed stringers, with 29 ft. clear span, substituted to carry the track, each chord of this truss being composed of three pine stringers of 8 x 16 in. x 31-ft. dimensions, trussed with two iron rods, 2 ins. square (see Plate XXXVIII), the coffer-dam showing no settlement whatever when the weight of the trains came upon it.

The entire cluster of cylinders was then cut down to about 2 ft. below the top of the coffer-dam; a pile-driver was set up over the coffer-dam, supported by trussed beams resting on the walls, the Lidgerwood pile-driver engine having been previously placed on the platform in the trestle, clear of the coffer-dam, in such a position that it could, with the aid of snatch-blocks, handle the four derricks as well as the pile-driver.

The pile-driver leads were 22 ins. apart in the clear, carrying a 3 000-lb. hammer. The piles used were of red cypress, 55 ft. long, with not less than 10 ins. of heart at the small end, and were driven with the assistance of a jet. The weight of the pile and hammer carried the piles down from 10 to 15 ft., while the jet was at full blast. The hammer was then used to strike, being raised only from 2 to 4 ft. at each blow, until a total penetration of 23 to 24 ft. was obtained below the cutting edge of the coffer-dam. The jet was then withdrawn, and a few blows, with a fall of 12 to 15 ft. were given, the piling all refusing to move under the last few blows.

The method of using the jet is shown below. It worked perfectly, the piles penetrating vertically, and in only two cases did they shoot out to one side, the workmen humorously terming them "flageolet piles." The tubes were old worn-out engine boiler tubes, taken from the scrap pile; the nozzle of the jet was 1½ ins. gas pipe, attached to 2 ins. jet hose, and of sufficient length to reach the working platform, where it was handled by one man, and raised when the pile was driven by means of a small line run through a snatch-block, and thence to one of the spools on the engine. The workman simply held the jet pipe firmly against the pile, and kept it pressed down inside the boiler tube, as shown on the accompanying drawing, while the pile was being driven.

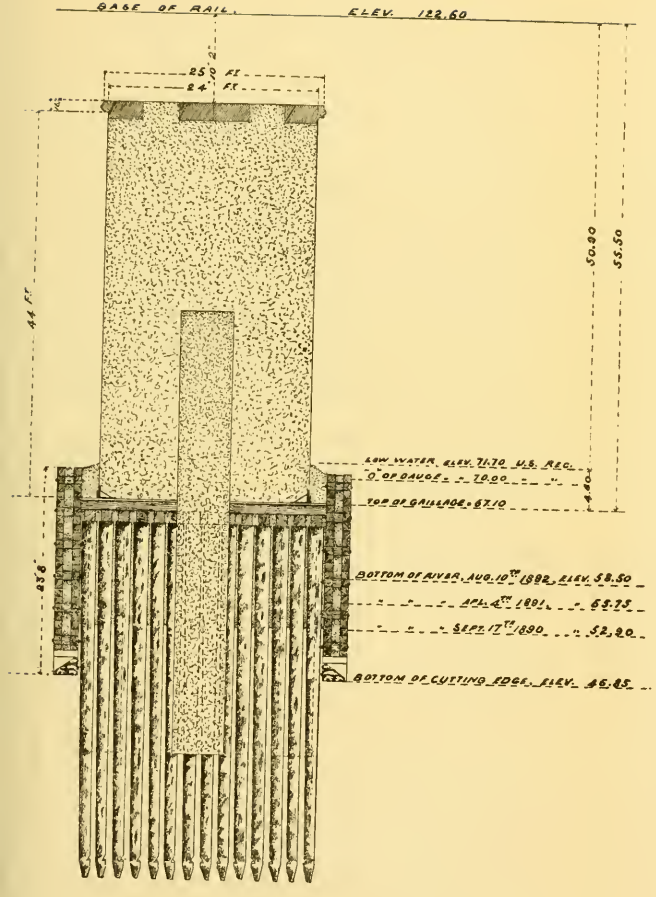


The piling having been previously skidded up alongside the track when unloaded from cars, two men with an axe and an augur were able to trim and notch the piles, insert the tubes and staple them to the piles fast enough to keep the driver supplied.

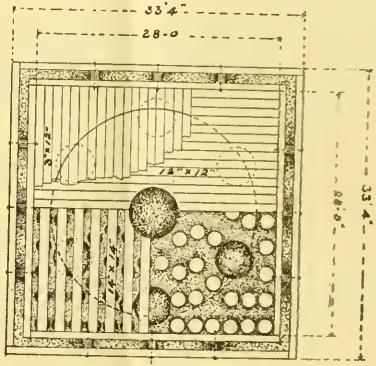
The work of driving commenced on May 29th, and was completed on June 13th. Two piles were the least number driven in any one day, and nine piles the greatest number; 87 piles were driven

PLATE XXXIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 695.
KELLEY ON RECONSTRUCTION OF PIVOT PIER.

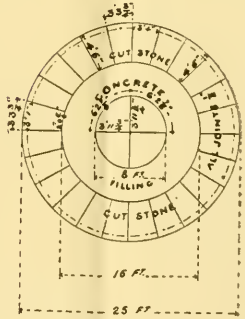
PIVOT PIER



SECTIONAL ELEVATION



PLAN



COPING

in all, or an average of $5\frac{44}{100}$ piles per day. The sand displaced by the driving rose to a height of 6 or 7 ft. above the cutting edge inside the coffer-dam.

The piles were then blocked into position for the grillage caps, after which a layer of Portland cement concrete 2 ft. thick was deposited around the piling. Over this Louisville cement concrete was placed to within 1 ft. of the proper height of the cut-off for the piles and the mass allowed to set for 24 hours.

A No. 5 Nye steam vacuum pump was then suspended in one corner of the coffer-dam, and on June 29th was set to work, pumping the coffer-dam dry in two hours. One or two small leaks developed through defective sealing in the corners of the bottom, while in the joints of the wall the leakage was not appreciable, most of the leaks being stopped by driving shingles in the joints; the total leakage did not amount to over 100 galls. per hour, an amount much less than had been anticipated.

No braces across the dam between the walls were necessary, for the walls actually stood a height of water of $16\frac{1}{2}$ ft. outside, without any bulging being detected.

The piling was now cut off ready for the grillage, the small cylinders 4 ft. in diameter were taken down to the same level, the concrete removed from them being thrown overboard for rip-rap; the entire bottom was then leveled off with good concrete even with the cut-off of the piles and the small cylinders.

The grillage consisted of a series of fourteen pine caps, 14 x 14 ins. x 28 ft., laid on the rows of piling and small cylinders, the center rows being cut to fit against the central cylinder which, it will be remembered, had not been cut down, but still projected up to within 2 ft. of the top of the coffer-dam. These caps were laid up and down stream and drift-bolted with round iron drift-bolts $\frac{7}{8} \times 22$ ins. Concrete was then filled in between these caps flush with their tops; cross-caps of pine 12×12 ins. $\times$ 28 ft. were then laid down on this, fitted to the central cylinder as before and drift-bolted to the lower caps. Upon this was laid a floor of 3×10-in. pine plank, laid up and down stream, and spiked with 8-in. boat spikes. The circumference of the 24-ft. cylinder shaft was then marked out on the floor, and the periphery of this circle brought to a perfect surface with the foot-adze; wrought-iron shoe braces were then bolted to the floor on the inner

circumference, to form a perfect circle for the bottom course of the cylinder shell.

This cylinder, as before stated, was 24 ft. in diameter, built of $\frac{5}{16}$-in. steel plate, 44 ft. high, in sections of 4 ft. in height.

When it was determined not to build the coffer-dam higher than 20 ft. on the gauge with the full penetration reached, it was decided to make the cylinder shell water-tight, so that concreting could be carried on inside it in the open air if the dam itself were submerged. The segments of each course were therefore made with planed horizontal butt joints and butt strips, the vertical joints of each segment breaking joints with those of the section below.

The bids for the cylinder material were based upon the following specifications, but the best bid received, calling for $\frac{5}{16}$-in. steel with a higher tensile strength, it was accepted and the steel cylinder used:

"Specifications for the Iron Cylinder Shell for the Pivot Pier at Ouachita River.

"The pivot pier shall consist of a wrought-iron riveted shell made of a good quality of plate iron $\frac{3}{8}$ in. in thickness, and 24 ft. inside diameter, and, when riveted up, shall be a total height of 44 ft.

"Each horizontal section of the shell shall be made of plates 4 ft. in width, and of such length that there may be twelve sheets of equal length in each horizontal section; they shall be butt-jointed in both horizontal and vertical joints, and riveted together with butt or packing strips on both vertical and horizontal joints, the packing strips to be of $\frac{3}{8}$-in. iron, 6 ins. wide.

"The horizontal strips shall be riveted on the outside of the horizontal joints; they shall be riveted in the shop to the upper edge of the sections; the vertical strips shall be riveted on the inside of the vertical joints, and shall project 3 ins. both above and below the plate, and shall be pierced with rivet holes on these extensions, so they may be riveted to the sections both above and below.

"All work to be single riveted with rivets $\frac{3}{4}$ in. in diameter and 3-in. pitch.

"All joints to be close and neatly fitted together, the horizontal joints to be planed to a true bearing so the cylinder may be caulked with a caulking tool after the sections are assembled together.

"To admit of easy handling, the sections must be shipped in third-circle segments one plate wide, and, to insure a close fit of the rivet holes for the field riveting, an inspector will be sent to the shops while the cylinder is being made and each horizontal section complete fitted

to the one above and below, and the rivet holes which are to be riveted in the field shall be reamed out to a perfect match; each segment must then be marked plainly with its exact position in the cylinder, so there may be no confusion of parts while the work is being assembled in the field.

"All riveting and shop work must be done in a thoroughly workmanlike manner, subject to the inspection and approval of the inspector.

"The iron used in the shell shall be capable of being bent cold to an angle of 180°, and to a curve whose diameter equals three times the thickness of the plate without showing signs of fracture, and shall develop a tensile strength, when tested in specimens of a uniform sectional area, of at least ½ sq. in. for a distance of 10 ins., of not less than 46 000 lbs. per square inch.

"The top and bottom section shall be reinforced on their top and bottom edges, respectively, with a regular 6-in. packing strip to secure greater stiffness.

"Enough rivets shall be furnished of the proper size by the contractor to complete the field riveting, allowing an excess of 10% over the actual number required.

"All segments shall be painted on both sides with one coat of good red iron ore paint, the quality of the paint to be subject to the approval of the inspector.

"Should it be found necessary, before the cylinder is completed and shipped from the shops, to add another 4-ft. ring or a fractional part in height thereof, it shall be furnished by the contractor in full conformity with the above specifications and at the same price per pound.

"Time being an essential object, the contractor will be obliged to state the time of delivery; all bids to be based on a price per pound f. o. b. Cairo, Ills."

The amount of shop work specified might be considered excessive, but as skilled labor of that class was difficult to obtain in this locality, it was thought advisable to secure accuracy in the shop; and the price per pound obtained was not in excess of the average price for ordinary cylinder material.

As fast as the cylinder segments were riveted in place, the interior was filled with concrete well rammed, the central 6-ft. cylinder being allowed to stand as it was and be entirely buried in concrete. The concrete for the cylinder shaft was made of the best English Portland cement with sand and broken stone, in the proportion of 1 part of cement, 3 parts of sand and 6 parts of broken stone, all measured by volume. It was mixed on a platform on the river bank in the usual

manner with hoes and shovels. From this platform an inclined track was built out on the side of the trestle, with a descending grade to the top of the coffer-dam. A push car carried a dump-box, which contained 1 cu. yd. of concrete, from the mixing platform to the coffer-dam, where the box was picked up by the derricks and dumped in any part of the cylinder required. The grade of the push-car track was such that when it was at the mixing platform, the top of the dump-box was level with the floor of the platform, so that nothing required to be lifted by hand.

On July 13th the first ring of the cylinder was riveted in position, and on August 28th the work was completed ready for the coping, some delay occurring, owing to the necessity of diverting the force to prepare the false work for the superstructure.

The design for the coping can be seen in Plate XXXIX. It consisted of a circular center 8 ft. in diameter, and a series of circumference or ring stones, the space between these ring stones and the center being filled with concrete well rammed to within 2 ins. of the surface of the coping, when the balance was filled up level and smoothed off with a mortar composed of Portland cement, sand and the screenings of the broken stone used for concrete, in about equal proportions. This made a very firm coping, and the concrete filling was hardly distinguishable from the cut stone, except for the absence of mortar-joints.

The coffer-dam having served its purpose, it was then torn down, coming apart quite readily with the aid of steel wedges and timber grab-hooks attached to the derrick falls.

The intention was to tear the walls down to within one or two courses of the top of the grillage, but, during the writer's absence, the foreman, mistaking his instructions, left four courses standing and filled the space between the walls and the cylinder with concrete, as shown in Plate XXXIX. This was allowed to remain, as it would have been an unnecessary expenditure of time and money to remove it.

The coffer-dam being a strong and permanent structure, that portion of it left surrounding the piling and penetrating the river bottom was a permanent and effective protection to the piling against scour and sinking drift, and was also a protection to the grillage; making, with its concrete filling of the walls and inside the coffer-dam, practically a broad caisson base upon which the cylinder shaft rested.

In reviewing the work it might be said that filling with concrete that portion of the wall space of the coffer-dam which was to be finally removed, was an unnecessary expense, and that clay would have answered quite as well; but the quantity of concrete being small, the security to be derived from the increased stiffness of the wall was considered ample justification for the expenditure.

During the entire progress of the work no trains were delayed to exceed 15 minutes, and this was of rare occurrence, being always in the case of an extra train or delayed freight, whose time of arrival was not known.

The confined space at all times hindered the work, and some unnecessary precaution may have been taken to prepare for possible emergencies; but those who are familiar with the rivers of the South-west will appreciate the necessity for an excess of caution, and it was a frequent occurrence for all hands to be called at night to break up and remove fields of drift which had lodged against the false work.

The total estimated weight above the grillage at extreme low water, assuming concrete to weigh 150 lbs. per cubic foot, is:

	Pounds.
Cylinder shell	67 124
Concrete and coping	3 076 024
Superstructure and train load	825 750
Total	3 968 898
Less buoyancy of submerged portion	130 257
Total net weight	3 838 641

or 4 896 lbs. per square foot of interior coffer-dam space. Deducting the weight supported by the cylinders of the old pier, and assuming that the balance of the weight is supported by the piling alone, it gives 38 288 lbs. as the weight supported by each pile in the coffer-dam.

Including all material used for coffer-dam, platforms, staging, barges and general work, the total cost of the work is shown by the following detailed statement, the entire accounts for labor and material having been kept separate from the regular accounts of the bridge and building department.

DETAILED STATEMENT OF MATERIAL.

Timber, 198 118 ft., B. M., at $13.98	$2 769 36
Piling—	
Cypress, 90 pieces, 55 ft. long, at .12 cents	606 00
Pine, 50 " 50 " at .06 "	150 00
Cement—	
English Portland, 1 000 bbls., at $2.48	2 480 00
Louisville, 462 " at .85 4/10 cents	394 90
Cylinder shell and rivets, 5/16-in. steel, 67 124 lbs., at 3.74 cts.	2 510 44
Iron shoe (plates and angles), 5/16-in. iron, 18 917 lbs., at 2.78 cents	525 27
Concrete stone, 750 cu. yds., at 75 cents	562 50
Rock (for concrete and rip-rap), 75 cars, at $5.886	441 45
Sand, 57 cars, at 80 cents	45 60
Clay, 35 cars, at $3.23 1/2	113 20
Coping stones	517 84
Oakum, 10 bales, at $2.68	26 80
Iron, spikes, bolts, etc	562 84
Tools	801 35
Sundry supplies and bills from other departments	685 21
Freight and transportation charges	612 75
Labor—including engineering and superintendence	11 703 06
Total	$25 508 57

Analyzing this statement, we find :

Original estimate	$30 000 00
Lowest bid	42 000 00
Highest bid	50 000 00
Actual cost by company forces	25 508 57

or $4 491 43 less than the estimate, and $16 491 43 less than the lowest bid received.

The labor represented 45 88/100% of the total cost; the tools, 3 14/100%, and the material and supplies, 50 98/100. The Hayward dredge and Nye pump were rented; the balance of the machinery and tools was either purchased new, or taken from stock in the Bridge and Building

Department, and all material and tools left over upon the completion of the work were taken back, and no credit given in the general accounts.

No sign of settlement or scour has ever occurred, but, on the contrary, when the false work was finally removed, the river commenced filling in around the pier, and the cross-section of the river now indicates a more stable channel than ever before.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

696.

(Vol. XXXI.—March, 1894.)

LINING A WATER-WORKS TUNNEL WITH CONCRETE.

By DESMOND FITZGERALD, M. Am. Soc. C. E.

READ FEBRUARY 7TH, 1894.

WITH DISCUSSION.

Description.—The Sudbury River Aqueduct, supplying Boston with water, extends from Farm Pond in South Framingham in an easterly direction to Chestnut Hill Reservoir in the suburbs of Boston, a distance of 15.9 miles. It is 9 ft. wide, 7 ft. 8 ins. high, and has a fall of 1 ft. to the mile. Its capacity is 100 000 000 galls. daily. Near the reservoir it passes through Beacon Street Tunnel in conglomerate rock, nearly one mile in length, and excavated in 1875. The conglomerate is crossed by strata of slate in places. This slate underlies the Boston basin, and is interrupted in the great Newton syncline either by an elevation above the present plane of erosion, or, as has been suggested by Professor Crosby, by a thrust fault. In 1877, it was found that disintegration had begun at a few points, and at this time a side wall 50 ft. long at Station 806, and 260 ft. of brick arching, in two separate sections, between Stations 810 and 813, were built in the tunnel.

The following is an extract from the report of Professor Raphael Pompelly, made at that time :

"The Beacon Street tunnel is in Roxbury conglomerate, and its axis is so nearly in the direction of the course of the bedding that where it strikes a narrow bed of the interstratified slate, this rises very gently, forming first the floor and one wall, then part of the other wall and roof, and finally disappearing above the tunnel roof. At several points this slate is much broken, and small quantities are liable to fall in, especially the north wall at 804 + 42, the south wall between 806 + 45 and 806 + 85, and the south wall at 810 + 50 to 810 + 80.

"But the nature of the roof and the character of the joints are such that there need be no fear that this will endanger the safety of the tunnel. There are only two points which seem to call for attention :

"*First.*—From 810 + 10 to 810 + 50 there are several joints in the roof running nearly in the direction of the tunnel and converging upward. The conglomerate has changed to-day along these seams to a depth of several inches; the decomposition of the rock and the position of the joints render the roof and walls insecure at this point, and I should recommend that this portion be arched.

"*Second.*—From 812 + 70 to 812 + 90 the roof has caved in to the height of 18 to 20 ft., owing to the horizontal columnar structure of the rock. I was not able to test the soundness of the rock in this dome, but it is not improbable that more rock may fall. The walls are good, and it seems to me that this might very properly be left undisturbed for the present. Should it be found necessary in the future to arch this point, the skew-backs could be cut in the walls above the level of the flowing water, and the arch could be built without interrupting the supply."

For several years afterwards small portions of the roof gave way, and it became evident that it was only a matter of time when it would be necessary to line portions of the tunnel.

In June, 1888, six cartloads of rock which had fallen were removed, and before the close of the year a large number of heavy pieces fell. It was decided to begin repairs at once, and before the increasing consumption of water in the city might form a serious obstacle to the undertaking. To show that the work was taken in hand none too soon, it may be added that 20 cartloads were removed in 1890, and that in December, 1890, during the work of lining, a mass of rock fully 10 tons in weight fell, demolishing the track and a switch, but fortunately injuring no one; other dangerous portions were braced up until they could be sustained permanently by the lining.

Preparations for the Work.—As it was necessary to maintain the

supply of water to the city during the construction of the lining, it was realized, before the plans were made, that the work would be difficult and would require careful thought in regard to every detail. Cross sections of the tunnel were taken, and it was found that in one place the roof had fallen to such an extent that it was 10 ft. above the average roof line. An estimate of the probable cost was based on lining at least 650 ft. of the dangerous portions, requiring about 2 500 cu. yds. of concrete. An outline of the scheme developed consisted in laying a track about 2 ft. above the bottom of the tunnel, taking the materials to form a lining of Portland cement concrete to the work in special cars, and mixing them close to the centers, the latter to be so designed that the cars, and also large flows of water, could pass through them.

The stations of the aqueduct begin with 0 at the westerly end, at Farm Pond, and extend in an easterly direction to the terminus at Station 838 + 32. The tunnel lies between Stations 776 + 65 and 823. There were three points of access to the aqueduct in the vicinity of the tunnel, one at Clarks' Waste Weir, Station 738 + 15; one at Station 772 + 80, near the westerly end of the tunnel, by means of a large gauging manhole; and the third by a manhole at Station 824 + 75, near the easterly end of the tunnel.

In June, 1889, shanties were erected at these two manholes, a ledge was opened in Newton Centre, a crusher set up, and in July about 50 tons of old rails, weighing 36 lbs. to the yard, were collected and delivered at Clarks' Waste Weir, which was the nearest opening of sufficient size to get them into the aqueduct. These rails were here straightened, drilled, where necessary, for the fish plates, and the switch rails curved to the proper radius.

The Track.—It was determined to make the track as perfect as possible, and it was fortunate that this was insisted on, for after several years' use it was still in as good condition as when first laid, and it never gave trouble. The gauge was 2 ft. 1¼ ins. When the rails were ready, the iron floor was removed from the gate house at Clarks' Waste Weir, and an inclined plane built leading down to the floor of the aqueduct. The rails were slid down to two cars prepared for the purpose. The iron wheels were covered with broad oak rims, which were beveled to conform to the invert of the aqueduct, and were also provided with guide wheels at the front and rear, projecting to the sides of the aqueduct, to keep the cars in the center. Twelve 30-ft. rails were trans-

ported at a load. On arriving at Station 772 + 30, the westerly end of the track, they were transferred to a two-wheeled cart, fitted with a long plank trough, carrying four rails to a load, and by means of these carts the rails were distributed along the line. The necessity for this transfer was on account of the change from the curved invert of the aqueduct to the level floor of the tunnel. The iron tires of these wheels were also covered with oak rims, to prevent injury to the concrete floor of the tunnel. This floor had been originally constructed by leveling up with débris and covering it with a 4-in. layer of concrete. The easterly end of the track was at Station 810 + 50. Length of track, 3 820 ft. There were four switches 50 ft. long, making total length of track and switches 4 020 ft. The average distance the rails were transported inside the aqueduct was 5 340 ft., and, when finally placed, they had been loaded and unloaded 13 times, and a portion of the work done in water 1 ft. deep, for it was not possible to wait until the water was all drained out of the aqueduct before beginning work.

The rails were supported on 500 trestle frames 8 ft. on centers. These trestles were made outside and transported by the same method used for the rails. The fall of the aqueduct in the length of the track was 8.7 ins. An additional pitch of 5 ins. was added in the construction of the trestles, giving 13.7 ins. total fall to the track, which made the pushing of the full cars an easy task. The trestles were built of 3 x 4-in. spruce joists. The two vertical supports were placed directly under the rails, and the union between the horizontal and vertical members was made by means of 1 x 3-in. strips, placed on both sides and fastened with 10-penny wire nails. There were also 1 x 3-in. pieces uniting the bottoms of the vertical joists. One-third of the trestle caps were made long enough to extend out to the sides of the tunnel, where they were wedged against the rock to keep the track immovable. The extremities of these pieces were braced down from above to keep the track from being floated when water was running. The other trestle caps were 5 ft. long. They carried a plank 2 x 10 ins. on each side of the track and two in the middle for the men to walk on. The trestles were provided with 1 x 3-in. diagonal braces, extending from the bottom of one trestle to the top of the next, and in reversed order on the other side. There were 10 000 ft., B. M., in the trestles, and 27 000 ft., B M., in the planking. The latter were floated from Clarks' Waste Weir. The rails were fastened to the trestles by means of a clasp 3 x 2 x $\frac{3}{16}$ ins. and

a $\frac{3}{8}$-in. bolt 4 ins. long, and to each other by the usual fish joints. The switches were placed at convenient points for passing, and where the tunnel was wide enough to permit of their construction. They were provided with red and green lanterns for signals. This elevated track allowed certain portions of the work to be carried on independently of the water running in the bottom of the tunnel. The cost was $3 324 32.

The Cars.—The cars were ten in number, seven for transportation of materials, two for transferring the concrete from the mixing beds to the lining, and one for miscellaneous use, such as moving centers, etc. The wheels were 20 ins. diameter inside the flanges, and 22.5 ins. on the flange. They were 2.5 ins. wide on the rail, and the flange was 1 in. thick, making total width 3.5 ins. Each wheel had six spokes. The axles were 2 ins. diameter, keyed solidly to the wheels. The bearings for the boxes were turned to 1.75 ins. The boxes were held in position by four bolts, two of which secured the upper part of the box to the frame work of the car and two others secured the lower half to the upper half; openings were provided for oiling the bearings.

The framework of the car was 5 x 1 ft. 9 ins. and was made of 3 x 6-in. and 3 x 4-in. oak material, and tenoned. A $\frac{5}{8}$-in. iron rod held the frame together at each end. An additional oak frame, 3 x 4 ins., was bolted to the frame proper to support the plank platform which consisted of three spruce joists 2 ft. 6 ins. long laid crosswise and planked with three 2 x 10-in. spruce planks 9 ft. 6 ins. long, projecting at each end of the car for the men to stand upon while unloading. The outside planks were bolted down, the center one being left loose for removal when oiling the bearings. Flaps of leather protected the oil holes.

The four cars which were to be used for the transportation of stone were provided with boxes 7 ft. 3 ins. long at the top and 4 ft. 9 ins. at the bottom, 2 ft. 6 ins. wide and 1 ft. 1 in. deep inside dimensions; capacity, level full, five casks of crushed stone. The ends of the boxes were made slanting to facilitate emptying. The three cars for the cement and sand were fitted with similar boxes, with the exception that they were 1 ft. 9 ins. high and were divided by a partition to separate the sand from the cement. These boxes held exactly two casks of cement and four casks of sand; they were made from spruce planks spiked together and fitted with two $\frac{1}{2}$-in. rods at each end, to prevent spreading. The ends were also set at an angle for convenience in unloading. The

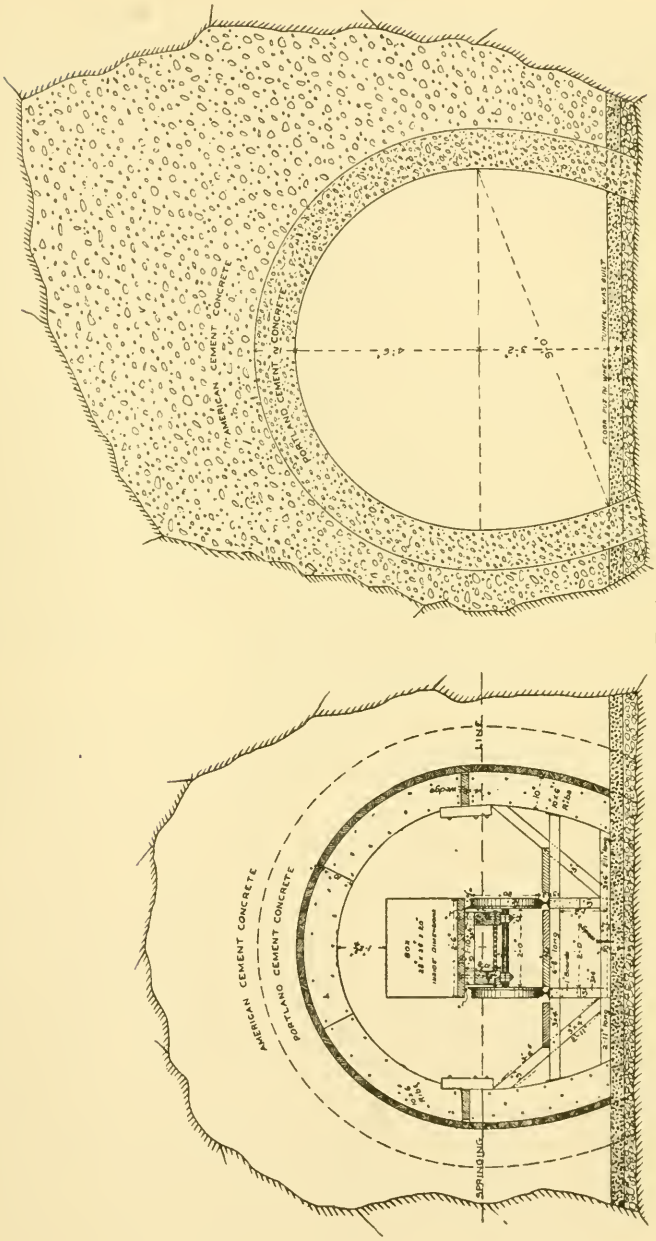

AMERICAN CEMENT CONCRETE
PORTLAND CEMENT CONCRETE
AMERICAN CEMENT CONCRETE
PORTLAND CEMENT CONCRETE

Fig. 1.

concrete boxes were 9 ft. 4 ins. x 3 ft. 10 ins. x 1 ft. 1 in. deep and of sufficient capacity for one batch of concrete mixed 1, 2 and 5 These cars had no ends. The sides were held in position by three iron bands passing beneath the bottom, extending up against the sides and and bolted to them. The cars cost $557 54.

Centers.—As no bracing could be used, the centers were designed of great strength. They were to be spaced 2 ft. on centers, but it was found that they were amply strong for 4-ft. spans, and they were so used. They were built of three thicknesses of spruce planks, breaking joints, and nailed with 4-in. wire nails. When completed they measured 5¼ ins. wide by 10 ins. deep. They were in three parts. The two side sections when in position extended from the bottom of the tunnel to 4 ins. above the springing line. The upper part was 6 ins. short of a full semi-circle. A 2-in. space allowed the introduction of wedges between the upper and the lower sections. The latter were triangular in elevation, with bases 3 ft. 8 ins. long, and braced diagonally. When set up the bases met in the center of the tunnel and were separated by a wedge. The side sections were united to the arch section by 3-in. oak blocks 5 x 14 ins., bolted by two ¾-in. bolts and one ⅝-in. bolt in the lower sections, and two ⅝-in. bolts in the upper center. Spruce board wedges 1 x 3 x 8 ins. were used in setting the forms.

There were 75 full centers containing 8 175 ft., B. M.

The lagging was made from 2 x 4-in. spruce joists in 8-ft. lengths, planed on both sides, with the edges beveled to fit the centers and secured with 10-penny wire nails. This lagging formed a tight cover over the centers. The total amount of lumber in centers and lagging was 14 000 ft., B. M., and the total cost was $1 460 55.

Shanties.—Sufficient shanty accommodation was not provided at first, but, as finally developed, the shanty at the westerly end, Station 772 + 80, was 100 ft. in length by 20 ft. wide, 7 ft. 7 ins. to the plate, with a lean-to on each side 12 x 22 ft. The frame was built of old materials, and the walls and roof of matched spruce boards. The roof was covered with tarred paper. It was found that the work of lining the tunnel could be carried on much better in the winter than in the summer, contrary to what was expected. As it was necessary for the laborers to wear heavy clothing for protection, and the heat from torches and lights was considerable, and the space for working very cramped, the men could carry on the work with less discomfort in the

winter when the air was comparatively fresh and cool and the ventilation good. After a little experience it was found perfectly feasible to supply materials for concrete free from frost even in the coldest weather. This, however, required considerable storage room in the shanty. The building was divided into compartments to contain 500 casks of cement, 160 cu. yds. of sand, and 80 cu. yds. of crushed stone and 80 cu. yds. of screenings from the gravel pit. At one end a large drying-room for the use of the men was provided. The gasoline and kerosene oil were kept in an out-building.

Shutes led vertically from the shanty down the manhole, terminating just over the cars beneath.

In the center of the sand compartment a large stove was surrounded by a deep plank bulkhead lined with iron, and the sand was piled all around it. By keeping up a fire in the coldest weather, the sand was kept free from frost.

The crusher was kept going at convenient times, and a large pile of crushed stone was kept on hand. It was covered with canvas, to keep out the rain and snow, and from time to time one of the lean-tos was filled from this pile and the other was filled with screened gravel. The screenings could not always be used on account of their arriving in a damp condition, when they were liable to freeze.

The cost of the shanties was $1 289 38.

Mixing Beds.—There were four mixing beds, each 15 ft. long, 5 ft. wide, and 10 ins. deep. They were built so that the boxes containing the materials on the cars projected over them, preventing waste of materials in loading and unloading. They were elevated above the floor of the tunnel on strong wooden horses and were built as high as possible, in order to reduce the lift to the smallest amount in shoveling the concrete into the cars. The widest and highest sections of the tunnel were selected for these mixing beds, and they were placed as near the work as possible. A board shelter covered with canvas kept them protected from the drip from the roof of the tunnel and they were securely braced. They contained 3 000 ft., B. M., and the cost was $303 47.

Equipment of Men.—Each man was furnished with a pair of long rubber boots, an oil jacket and a lantern. The articles were all numbered, and a record kept, with the name of the workman having them in possession.

Construction.—As work inside the tunnel could only be carried on for four days in the week, and a large flow of water had to be run during the other three days, to make good the loss in the reservoir, the work of laying the track occupied the greater part of the summer. In September the shanty was stocked with cement, sand and crushed stone, and on October 15th the actual work of construction began.

The first work consisted in preparing the bottom. It was not thought wise to build the lining on top of the concrete floor with the dry filling underneath. To cut all the sides out would have cost a large amount of money, so piers were cut out 8 ft. long with an 8-ft. space between them, and breaking joints on each side of the tunnel. The faces of the piers were stopped 2 ins. short of the face of the lining, thus breaking the joint over the edge of the opening. All the filling under the concrete floor, consisting of broken stone, gravel, sand and clay, was removed to the solid rock.

As the water was always from 3 to 6 ins. in depth on the floor of the tunnel, it was necessary to lay sand bags around these excavations. The water came in so rapidly as to require constant pumping. The average pier contained 16 cu. ft. of concrete.

A dam, to regulate the water in the tunnel and keep it from flooding the work, was placed at Station 782, west of the work of repairs. The top of this dam was 2 ins. below the bottoms of the rails in the track. This dam accumulated the waters of percolation until they were raised sufficiently high to reverse the flow under the track, allowing it to escape at Clarks' Waste Weir. By this means, also, an abundant supply of water was always at hand for making the concrete. The dam was fitted with two large valves. When desired, a second dam was placed easterly from the first and located between the mixing beds and that part of the tunnel where the concrete was being deposited. The water was thus kept at its lowest point where the centers were being placed. From the second dam the in-filtering water could drain off freely to a plug connecting with a sewer at the terminal gatehouse at the reservoir. The valves were partially opened at night, to prevent an overflow during the day; and were taken out when the flow of water was let on at the end of the four days' work.

Disposition of the Force.—Two men were kept at the bank screening sand and preparing stone and screenings, one man in the building shoveling sand, two men shoveling crushed stone, one man opening

cement, spreading it upon the floor and picking out the paper and shoveling it down the slide. This man also attended to the gasoline torches and lanterns for the switches; he also carried the torches to the work in the morning and back at night. There were seven men on the cars and sometimes eight, transporting the materials to the mixing beds. A foreman and five men were stationed at the mixing beds, two men ran the concrete cars from the beds to the work; two additional men helped unload; four men were employed in laying and ramming; four men digging the foundation piers, removing débris, pumping water, washing the tunnel, and preparing the centers; three men taking down centers and setting them up and helping carpenters; three men taking nails out of lagging, scraping and washing it, and three carpenters setting centers, building drains, weepers and bulkheads. Whole number of men, including superintendent, 41 men for four days, weekly. On the remaining two days in the week, the aqueduct being then half full of water, a portion of the force was occupied in crushing stone and filling the supply buildings, and the remainder, with the carpenters, were provided with work elsewhere.

Portland Cement Concrete.—As the lining was to consist wholly of concrete, it was of the utmost importance that it should be of good quality. Some experiments were made to determine the actual proportions of the materials as used. It was found that in dropping 12 ft. down the spouts to the car, some consolidation of the crushed stone and sand took place. As two casks of cement were always emptied down the spout into each car, irrespective of the space they occupied, these two casks remained unchanged, but rather more than four casks of sand were shoveled down, to fill the sand compartment of the car. Two casks of sand shoveled into a measuring box at the surface of the ground were found to occupy a space of 6.84 cu. ft., and in the car (12-ft. drop) 6.37 cu. ft., a shrinkage of 7 per cent. Five casks of crushed stone, well screened, filled 16.87 cu. ft. at the surface, and in the car 15.31 cu. ft.; a shrinkage of 9 per cent. Consequently, as enough material was shoveled down to fill the car, we have 18.56 cu. ft. of crushed stone and 7.35 cu. ft. of sand used with every cask of cement, making, when taken out of the car and spread in the mixing beds, a proportion of one cask of cement, as it came from the dealer, two and one-sixth casks of loose sand, and five and one-half casks of loose crushed stone. By slightly shaking the sand and stone in the casks it was found that

this amounted practically to the proportions of 1, 2 and 5. By actual measure this was found to make 20 cu. ft. of concrete when thoroughly rammed. The crushed stone was somewhat smaller than the average size. The concrete was of exceptionally good quality, as was determined by inspection of large sample blocks. One of these blocks, containing 20 cu. ft., was drilled through in three places, and opened by means of long steel wedges driven from two sides, and found to be perfectly solid and free from voids.

The concrete was hardly compacted as solidly in the tunnel, and there it occupied 21 cu. ft. per cask of cement, 1.3 casks of cement laid in 1 cu. yd. of concrete.

Mixing.—Each batch of concrete was mixed by spreading five and one-half loose casks of crushed stone evenly to a depth of 5 ins. in the mixing bed. One cask of cement was then spread over the stone, and two and one-sixth casks of coarse bank sand on top of the cement. These materials were then mixed, in a dry condition, the men being required to show the backs of their shovels turned evenly upwards at each cast. This part of the mixing was done in advance by one man, while a wet mixing was going on in another bed close by. The wet mixing which came next was performed by four men. One man applied the water, while one man on each side turned the concrete towards the center of the bed, while the fourth man behind the others did the same. The concrete, after having thus received one dry and one wet mixing was then shoveled into the two concrete cars, which were then run to the work, one man to each car. On arriving, another man was detailed to each car, and the concrete was shoveled into place.

The four men who did the laying and ramming were obliged to use great care to insure thorough work, for the coarser portions always had, as usual, a tendency to fall against the lagging. If these had remained, the exposed surface of the arch, on removal of the lagging, would have been full of cavities, and it would have been almost impossible to have resurfaced the work.

It was originally intended to use Portland cement concrete for a distance only of 1 ft. from the lagging, and to back it up with American cement concrete. Where there was a considerable space to fill, this was practicable, but ordinarily it was found cheaper to use Portland

PLATE XL.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 696.
FITZGERALD ON CONCRETE TUNNEL LINING.

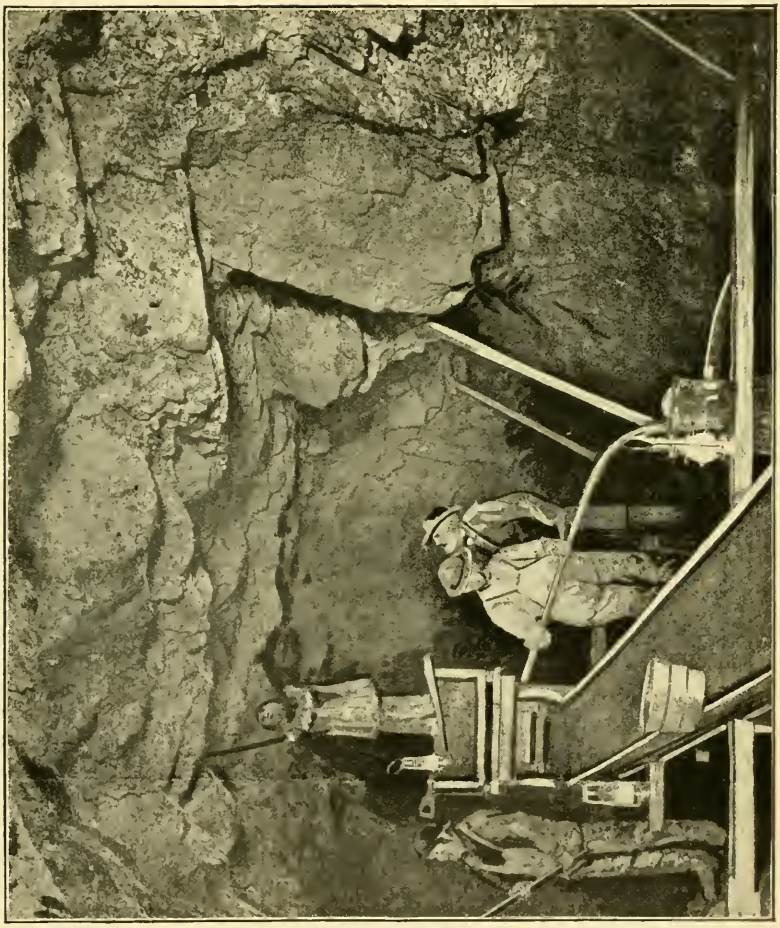


PLATE XLI.
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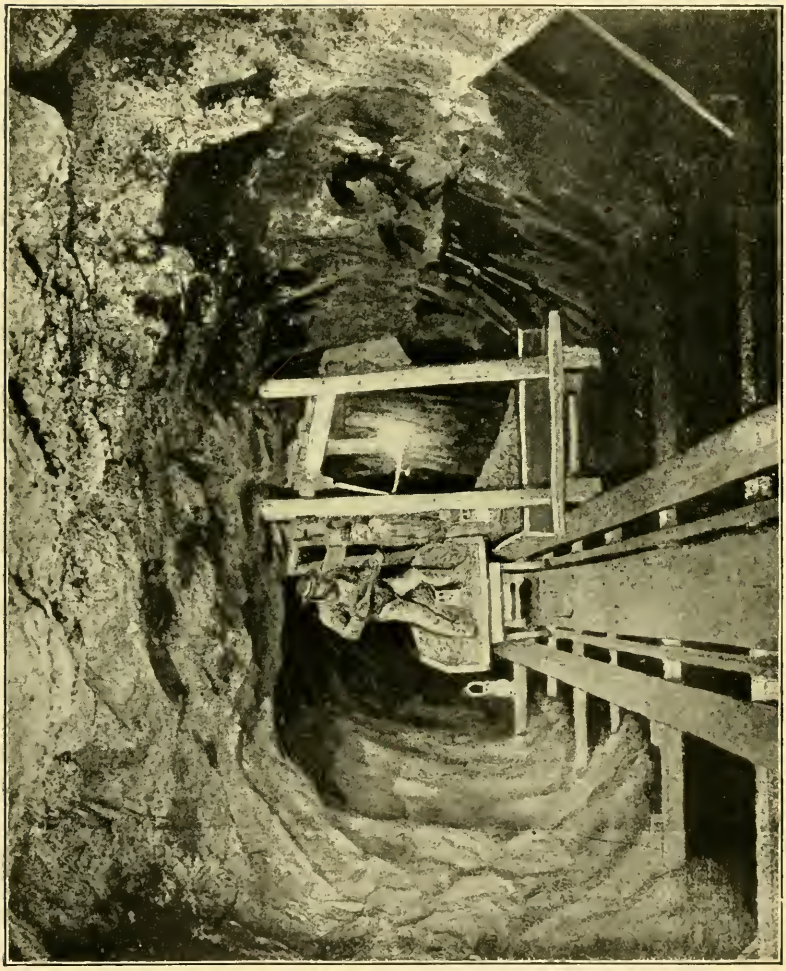

PLATE XLII.
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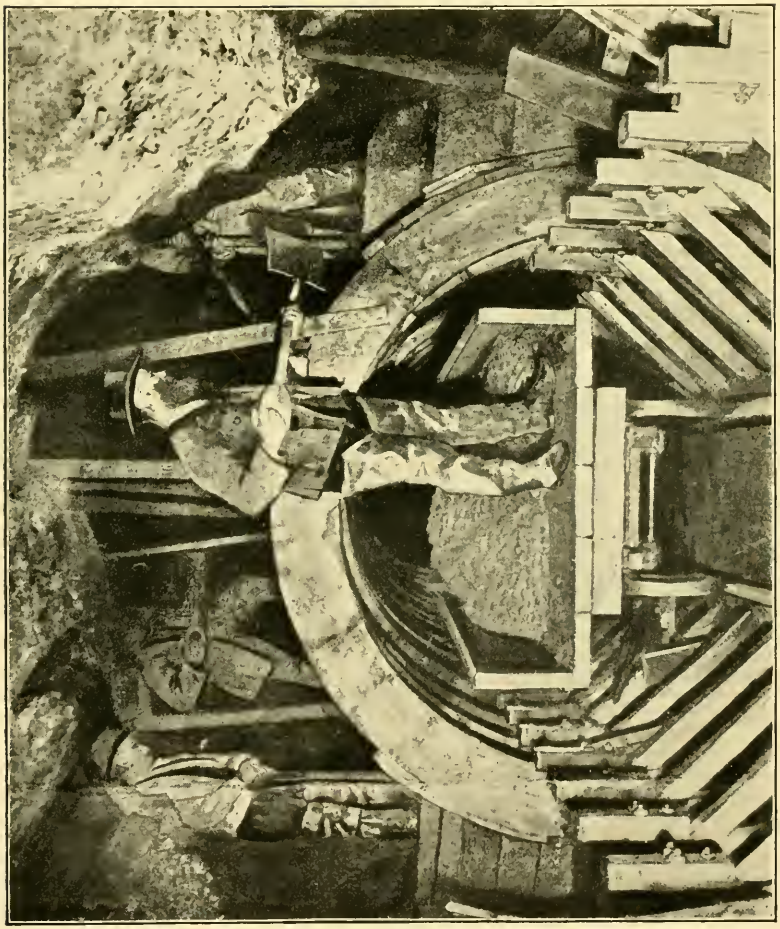


cement entirely, owing to delays, lack of room and other difficulties. The different portions of the work were so closely related that a stoppage in one place caused a disarrangement of the whole system. When changing from one cement to the other on the sides, it was necessary to build bulkheads, and these had to remain in place until the concrete had set. The filling of the space near the top of the roof of the tunnel was the most difficult part of the work, as the men had to work in a crouching attitude and push the concrete with long-handled implements into all the crevices.

The average number of casks of material put into the work daily was 192. This was transported an average distance of 2 700 ft. Seven men did this work; one load consisted of one car of stone and one car with the cement and sand, and this load contained 11 casks. Three men pushed these cars, and made eight trips per day (88 casks). The second gang of three men made seven trips per day (77 casks), and the gangs alternated daily. An additional stone car was run by one man, averaging 27 casks daily. Distance traveled per man per day propelling cars, 7.66 miles. Each man also shoveled from the cars 27.4 casks of material, spreading it evenly into the mixing beds. The loaded cars had the right of way. The returning cars were obliged to make quick trips from switch to switch.

Tools.—The concrete was put in place with shovels and rammed with rammers of various forms, the principal ones being made from oak timber 5 ins. square, 12 ins. long, with a 2-in. handle, 30 ins. long, inserted in one end. The concrete was pressed into the cavities in the roof with long-handled T-shaped implements, the iron portion being 3×8 ins.

Weepers.—Large quantities of water poured from the roof in rainy weather in certain places, and it was found necessary to build many weeping drains into the concrete. They were built of spruce boards, 6 ins. wide, in 4-ft. lengths, and were placed on both sides of the tunnel, 16 ft. apart. The first lengths were laid on the bottom, extending through the centers and cut off flush with the lining. Vertical sections fitted against the walls connecting with the bottom sections. The sides of these boxes next the walls were open, with the exception of the first vertical section and the horizontal section, which were tight, to prevent the escape of cement.

Bulkheads.—Bulkheads were usually 16 ft. apart lengthwise of the tunnel. They held the concrete in place while it was rammed and during setting. Bulkheads for the arch were 4 ft. apart. Steps were formed in each section of the concrete lining by means of bulkheads from 1 to 2 ft. wide and 20 ins. high. A bond was thus formed between the sections.

Arrangements for Running Water for the Supply of the City.—The water was shut off from the aqueduct on Sunday at 3 P. M. At 4 A. M., Monday, the gates at Fuller's Waste Weir, Station 459, were opened, and also at Clarks' Waste Weir, Station 738.

The depth of the water remaining in the aqueduct at this time was generally 7 ins. at Fuller's, and 16 ins. at Clarks'. Two stop planks across the aqueduct between the two gates at Clarks' Waste Weir separated the water wasting from the west from that wasting from the east which came from the dam at Station 782. Easterly from this dam the water escaped into the reservoir. At 7 A. M., Monday, the connection with the reservoir was closed and the plug leading to the sewer opened. Work in the tunnel was started at this time and prosecuted until Thursday evening.

At 2 P. M., Thursday, a flow of 50 000 000 galls. daily was started, arriving at the tunnel after the men were all out. This flow continued until Sunday at 2 P. M. It reached to within 1 ft. of the underside of semi-circular form. The gates at Fuller's were closed on Tuesday at 2 P. M. and at Clarks' at 5.30 on Thursday. The plug was put into the sewer at 10 P. M. and the water arrived at the reservoir at 11 P. M.

To prepare for this large flow, the cars were loaded with materials and side-tracked, other cars ballasted with sand bags and left upon the main track in a wide section of tunnel, centering braced down, lumber and tools scaffolded, chips and waste material gathered, and the sanitary (earth closet) removed.

Progress of the Work.—One thousand one hundred and eighty-two linear feet of tunnel lining were laid in portions of three different winters as follows:

October 15th, 1889, to April 18th, 1890.....	562 ft.
January 14th, 1891, to May 7th, 1891.....	340 "
February 1st, 1892, to April 6th, 1892.....	280 "

The following table of statistics refers to the three working seasons in order:

Season.	Days.	Linear feet.	Linear feet laid daily.	Cement. Number of casks. Portland.	American.	Total.	Average per day.	Concrete. Number of cubic yards. Total.	Average daily.	Cubic yards per foot.	Expenditures. Total expenditures.	Price per cubic yard.
1st.....	89	562	6.20	1 715	349	2 064	23.19	1 605	18.04	2.85	$29 272 50	$18 24
2d	59	340	5.76	1 245	76	1 321	22.39	1 027	17.41	3.00	13 990 59	13 62
3d	38	280	7.37	1 009	61	1 070	28.00	832	22.00	2.98	12 679 63	15 24
	186	1 182	6.36	3 969	486	4 455	23.95	3 464	18.62	2.93	$55 942 70	$16 15

Cost.—The total expenditure on account of the lining of the tunnel, including cleaning up outside and inside after the work was completed, was $55 942 70. This expense included some work which might justly have been charged to other appropriations; but, on the other hand, there are always some expenses under conditions similar to these which do not get into a force account, however methodically kept. This makes the cost per cubic yard $16 15, which the writer believes to be moderate in consideration of all the circumstances. The track is now in the tunnel, ready to be used again, should any other portions of the tunnel require repairs in the future. If the taking out of the track is to be charged to the work already done, the price named above is not quite sufficient.

The foreman in charge of construction was Mr. J. W. Oldham, and the writer desires to record here his appreciation of the skill and fidelity with which he superintended and executed the work.

DISCUSSION.

R. W. Lesley, Assoc. Am. Soc. C. E.—I should be glad to know if any record was kept of the proportion of water used, and what proportion of sand and broken stone went to make the concrete. There have been so many different figures on this, so that recently a leading

contractor took a contract for concrete in which he figured that he was going to use so many parts sand, so many parts stone, so many parts cement, etc., and that as the sum of these materials made so many cubic feet it would cost him so much per square. As he had omitted to figure that the cement and sand filled the voids in this stone he came $33\frac{1}{3}\%$ short on what he expected to get for his work. As these are the first figures I have seen on this subject I would be very glad to know if there were any figures on the water, so that we would know how much of the various ingredients would make a yard of concrete. It is a very interesting problem, now, to determine exactly what is required to make one yard of concrete of a certain proportion.

Mr. FitzGerald.—A great many accounts of concrete ingredients have been published. I regret to say that we did not keep any account of the water, but my instructions were to mix the concrete for each batch as dry as it could be worked, and to ram it very thoroughly.

Mr. Lesley.—The concrete, as it was rammed, brought the water to the surface, as I understand. The figures, according to your paper, would give about 30 cu. ft. of material, excluding water; a barrel of cement is equivalent to about 4 cu. ft.

Mr. FitzGerald.—Yes; 29.4 cu. ft. made 21 cu. ft. of concrete with one cask of cement, or 38 cu. ft. of materials made one cubic yard of concrete with one and one-third casks of cement. A singular fact is that if you get a pile of broken stone together and measure the voids carefully and put in enough cement and sand to fill those voids, it more than fills them.

Mr. Lesley.—I have made some experiments to determine the water in concrete. The great question in the experiments I conducted was the porosity of the lime stones and trap rocks we used. I found from our experience the only way was to make a box of a given size and actually determine what went into the box, each rock requiring different proportions of water. I see you took the run of the crusher.

Mr. FitzGerald.—True, it was not passed through a ring, but it was all screened through a circular screen to a definite size.

Mr. Lesley.—The figures are very interesting and the first I have seen on that particular question.

Mr. FitzGerald.—I have had some blocks of concrete mixed containing about 1 cu. yd. These were broken open by boring three holes clear through them in the same plane and inserting long, thin steel wedges. The blocks were found to be perfectly solid and free from voids, but how water-tight I cannot state.

Mr. Lesley.—On the subject of the permeability of mortar there were some figures made and published by the Engineering Department of the University of Pennsylvania, giving the permeability of various mortars and cements.

Mr. FitzGerald.—How did they get the porosity?

Mr. Lesley.—They took little cylinders of the material and then put on the pressure from an ordinary water pipe, the discs were put in the water pipe and the pressure was allowed to go through them. I have that record on some twenty or thirty different materials.

Charles B. Brush, M. Am. Soc. C. E.—I am very glad that Mr. FitzGerald has given us the results of his work in such detail. I feel especially interested in this paper because I personally urged Mr. FitzGerald to present it to the Society. There are several question, I should like to ask. What was the number of cubic feet in a cask? I notice that those two casks of sand shoveled in the measuring boxes show that two casks of sand occupied a space of 6.84 cu. ft., and five casks of crushed stone 16.87 cu. ft. This would seem to indicate that it was 3.42 cu. ft., and the five casks of crushed stone will give 3.37.

Mr. FitzGerald.—I am much obliged to Mr. Brush for calling attention to this matter. The measurements given were averages of a number of measurements, and without any attempt to fudge them to an agreement when reduced to the same unit. The contents of a cask were accurately determined, however, by putting it into a box surrounded with concrete and then filling the cask with water. It took just 25 galls., 2 qts. and 1 pt. to fill the cask between the heads, and this gives us by calculation 3.425 cu. ft., which represents the quantity of cement and paper as they came from the manufacturer. That the measurement of the sand (see page 303) should have agreed with this exactly is a mere accident. It should really have measured a little more, because one head was out of the cask. The sand, after being filled into the cask, was taken out, and, for tests, measured in a rectangular box. The same was done with the crushed stone, and that one measured 3.42, and the other 3.37 when reduced, shows how carefully the measurements were made.

Mr. Brush.—I would like to ask what brand of cement was used in this work; whether the same brand was used throughout or whether different brands were used?

Mr. FitzGerald.—We used mostly Brooks, Shoobridge & Co.'s cement. It gave $36\frac{7}{10}\%$ retained by No. 120 sieve* and, with $22\frac{1}{2}\%$ of water, gave an average tensile strength of 388 lbs. neat, and, with 2 parts sand and $13\frac{1}{2}\%$ of water, gave 51 lbs., and, with 3 parts of sand, 12% water, gave 21 lbs., and with sand, 3, cement, 1, 312 lbs. in compression. All seven-day tests.

Another series of tests gave $30\frac{1}{2}\%$ retained by sieve 120, and 182 lbs. tensile strength neat cement, 24 hours, 22% water. Sand, 1, cement, 1, one day, 65 lbs., 14% water, and 250 lbs. at the end of seven days. With 3 of sand and 12% water, 90 lbs., seven days, 147 lbs., 28 days, and 380 lbs., per square inch compression strength, seven days.

Mr. Brush.—A slow-setting cement?

* 106 and 116 meshes to the inch each way by actual count.

Mr. FitzGerald.—Rather slow, about 50 minutes to take the heavy wires; slower than the Alsen, but not nearly as slow as the German and French cements, which take sometimes seven hours to bear the heavy wire.

Mr. Brush.—Referring to the diagram, it would seem that at places there was a very large space between the arch and the rock excavation.

Mr. FitzGerald.—That is where the roof had fallen.

Mr. Brush.—About what was the average over the arch?

Mr. FitzGerald.—I don't think the average would be over 2½ ft.

Mr. Brush.—As I understood you when you read the paper, the filling in was all done with Portland cement instead of American cement, except in the deep pockets.

Mr. FitzGerald.—Yes.

Mr. Brush.—I would like to ask whether the results might not have been obtained equally well by using dry stone filling instead of concrete filling; whether it would not have largely reduced the cost.

Mr. FitzGerald.—Undoubtedly it might have reduced the expense, but we thought it would be better to make the work homogeneous, as those deep pockets were not very long. There could then be no question about the permanence of the repairs. The percolation of water might have brought enough oxygen down with it (especially in the winter when the water contains so much more oxygen) to have produced some farther disintegration; still that is a conjecture.

Mr. Brush.—Was the percolation considerable?

Mr. FitzGerald.—It was in streams in some cases.

Mr. Brush.—Where you struck it in streams, did you convey it directly into the drains?

Mr. FitzGerald.—Yes, and the drains emptied into the aqueduct. That leads to a very interesting question if you follow that up.

Mr. Brush.—I was leading up to that question. I would like very much to know your views on that subject.

Mr. FitzGerald.—I do not believe very much in building aqueducts below the levels of the water tables of the country through which they pass. I think an aqueduct to convey drinking water should not do this, because I know from many years of experience there are places where a great deal of contamination comes from the ground-water flow into an aqueduct. It is one of the elements that I think a man ought to consider in designing new work.

Mr. Brush.—The dry stone lining under the concrete floor was, I suppose, the débris that was in the tunnel?

Mr. FitzGerald.—Yes.

Mr. Brush.—And then this floor was laid solid from the face of the rock on each side of the excavation?

Mr. FitzGerald.—Yes.

Mr. Brush.—And the arch was sprung from there?

Mr. FitzGerald.—For a length of 8 ft. the new work was carried down to the solid rock, then for the next 8 ft. it rested on the floor, alternating in this way in the length of the tunnel.

Mr. Brush.—The floor, then, was already in ?

Mr. FitzGerald.—Yes, and in order to get a good foundation for the lining I took the floor out in the shape of these piers, and connected with the solid rock at the bottom.

Mr. Brush.—Did you find the flooring in good condition ?

Mr. FitzGerald.—Yes; but you must bear in mind that there was dry filling under the thin concrete floor. I was not willing to build wholly on top of this floor.

Mr. Brush.—You will, perhaps, leave this staging in the aqueduct some time yet ?

Mr. FitzGerald.—No; as soon as we got one section laid, we kept pulling down the centers back of us and building them forward.

Mr. Brush.—Did you not pull down the platform ?

Mr. FitzGerald.—No; the track was entirely independent of everything. We did pull up our track when we had 2 000 ft. laid, removing all track up to that point; the tunnel was completed then.

Mr. Brush.—How long after you finished any portion of this arch did the water come in contact with it ?

Mr. FitzGerald.—On some portions almost immediately; on others within three days. If you give it about 24 hours set, then the fine finish at the surface will remain; but if you allow the water to run over cement work within 24 hours, the body of the cement will stand, but the fine gloss disappears from the surface.

Mr. Brush.—In laying this concrete, what thickness did you make your layers?

Mr. FitzGerald.—We kept it in one homogeneous mass, going right along and filling it in.

Mr. Brush.—Ramming it up against the sides?

Mr. FitzGerald.—Yes; stepping it up.

Mr. Brush.—Going along the back, did you keep the layers horizontal?

Mr. FitzGerald.—Yes; of course we had to vary them a little, and to build a great many bulkheads.

P. F. Brendlinger, M. Am. Soc. C. E.—Mr. FitzGerald says: "In 1877, it was found that disintegration had begun at a few points, and at this time a side wall, 50 ft. long, at Station 806, and 260 ft. of brick arching, in two separate sections, between Stations 810 and 813, were built in the tunnel." What was the objection to continuing the brick arch in this place where there was a concrete arch put in?

Mr. FitzGerald.—The brick work would have cost a great deal more money; that is the only reason. I designed originally to make this arch of brick, and back it up with concrete.

Mr. Brendlinger.—Why back it up with concrete? In a contract for brick work in a tunnel now building, the cost is about $8 a cubic foot; this cost $16.

Mr. FitzGerald.—The cost was owing to the conditions. The brick work would have cost still more; the cost of this work came from the conditions under which it was performed.

Mr. Brendlinger.—The New York Aqueduct was built in various types of section, but I don't know why. The arch was all brick work, and the rubble masonry backing was carried up to about half way between the springing line and the center. The remaining space was filled up with dry packing to a thickness of 4 ft., in some cases. Now, it seems to me that under the circumstances and conditions you worked under, a brick tunnel could have been built much cheaper than the concrete. As to the question of getting material in, you can make eight trips a day; it is not one mile to haul.

Mr. FitzGerald.—The tunnel was about one mile long.

Mr. Brendlinger.—Eight trips a day is not much; some of the men went but seven trips a day; that could not have been much more than half a mile.

Mr. FitzGerald.—I think I have given the exact distances, but, of course, you can get them from the stations. It must be remembered that, as stated in the paper, these men who pushed the cars did other work.

Mr. Brendlinger.—It looks to me as if the material could be put in there sufficiently fast so that there could be probably 30 ft. of arching completed in 24 hours, in brick work.

Mr. FitzGerald.—We allow for concrete $6 00 per cubic yard, and you cannot do brick work for anything like that.

Mr. Brendlinger.—You can build brick work for $10.

Mr. FitzGerald.—I don't think we have built it quite as cheaply as that in Boston, even by contract.

Mr. Brendlinger.—Those are contract prices. I know a contractor can do work faster and cheaper than an engineer.

There is one thing about getting in your track. You say in the first place they took down twelve rails, and put them in a wagon or car, and transported them to a certain point, and then put them on a cart, and they handled them thirteen times. Why was it necessary to handle them more than once?

Mr. FitzGerald.—There was a difference in sections; the tunnel was of an entirely different section from that in the aqueduct. This car was made to fit the invert of the aqueduct. The iron wheels were bound with oak tires, and that same car could not have been run through the tunnel.

Mr. Brendlinger.—The tunnel itself was not lined with brick work at the bottom?

Mr. FitzGerald.—No; the tunnel was of irregular section with a straight floor.

Mr. Brendlinger.—The handling includes everything, from the outside?

Mr. FitzGerald.—Everything.

Mr. Brendlinger.—It looked as though there was more handling than necessary.

Mr. FitzGerald.—It might appear so, but if you count every time a man handles a rail it counts up very fast.

Mr. Brendlinger.—In regard to the track, it struck me that there could be a better arrangement made. Of course, you have spent years on it, and I have just read the paper once.

It struck me that there must have been considerable resistance or interference with the flow of water, and there might be a different arrangement made.

Mr. FitzGerald.—We found in transporting the material, which was all pretty heavy and had to be carried very fast, that it was a great thing to have the track very rigid. The supports were directly under the rails.

In regard to the flow of water, there was plenty of room between the centers for a flow of 60 000 000 galls. per day, so there was ample space. The centers were all built without any bracing.

Mr. Brendlinger.—You supported the roof entirely from the centers?

Mr. FitzGerald.—Yes.

Mr. Brendlinger.—Was there much rock falling?

Mr. FitzGerald.—Not a great deal.

Mr. Brendlinger.—Judging from the speed you made, I thought you could not have much rock falling. Using dry filling would have reduced the cost to $8 a yard, instead of $16.

Mr. FitzGerald.—Oh, no; the ordinary distance from the top of the arch was only $2\frac{1}{2}$ ft., and, in places, less than that. I am sure it would have cost more to have stopped our concreting for this small amount of dry filling.

Mr. Brendlinger.—But I am suggesting that no concrete at all be used.

Mr. FitzGerald.—You mean brick work?

Mr. Brendlinger.—Brick work.

Mr. FitzGerald.—The only reason we took up the concrete was on account of the cheapness, but this is a matter on which men may well differ, and I wish to hear some reasons in favor of the brick work. Of course, if dry filling is used I can understand its effect in reducing cost, but would this have prevented farther disintegration?

Mr. Brendlinger.—In connection with this subject, I might say that in the repair of the Croton Aqueduct we used a similar construc-

tion. The wagons had 6-in. wooden wheels. The wagons would fill a barrel full, and each wagon held nine barrels of cement and sand mixed. Of course, there was a little injury done to the invert, but that was done by the mule shoes.

James Owen, M. Am. Soc. C. E.—How long after the concrete was set in place did you strike the centers? What period of rest did you give?

Mr. FitzGerald.—That varied with the convenience of the work. We had 75 centers and we kept moving them up from the end of the work. Sometimes they would stay in quite a long time; at others, not a week. I should say there was no trouble about taking them down a few days after the concrete had set. We did this work when we had an opportunity, in winter. We would put the men in and work them for four or five weeks or more on the work, and sometimes the centers were in quite a while before being struck; at other times we wanted the centers and we would take them down and use them.

Mr. Owen.—You did not find any trouble with water flowing through the roof?

Mr. FitzGerald.—No; because that was all carried down through the drains. The drains were a very essential part of the work; they were cheap affairs, simply made out of spruce boards.

William Barclay Parsons, M. Am. Soc. C. E.—I am very glad that Mr. FitzGerald has presented this paper to the Society. Engineers, at least in the eastern part of our country where there is a widely distributed abundance of good building stone, have not given enough attention to stone's humbler rivals—brick and concrete. Encouragement should, therefore, be given to every paper presenting the merits of either of the latter. For linings, such as Mr. FitzGerald describes, concrete is particularly well adapted.

A few months ago I had occasion to inspect professionally some of the large irrigation works in California and Arizona, and was much impressed by the successful, ingenious and bold manner in which concrete had been used for the lining of both canals and tunnels. Irrigation tunnels which resemble the tunnel described by Mr. FitzGerald are generally lined with concrete from 4 to 6 ins. thick, put in place by the aid of a track and a movable center.

One of the great advantages possessed by concrete as a lining for tunnels is that greater strength is obtained for the same thickness of ring than with brick. If the latter is used, the backing must be either of concrete or dry packing, which, being a separate material from the brick, adds nothing to its strength, so that the lining proper has to be strong enough to resist the whole pressure, and with brick the arching would have to be at least 8 ins. or two bricks thick. With concrete, however, the lining and backing form one homogeneous mass, so that in tunnels of small section it is safe, unless the material passed through

is of a very treacherous nature, to use a lining of a thickness of 4 or 6 ins., as compared with 8 ins. of brick, outside of projecting points of rock. In lining old tunnels, as in the Boston case, where it is undesirable to encroach on the sectional area, this is a great advantage.

Irrigation canals in California are also lined with concrete, which is put on in three ways: Either as a regular concrete, composed of broken stone or gravel; or else the bottom is roughly paved with smooth field stones to a depth of about 8 ins., a lime mortar being poured in to fill up the interstices and a strong Portland cement mortar flushed over the whole, the lining of the sides, generally about 4 ins. thick, being carried up from this foundation; or, thirdly, in some cases the sides and bottom are carefully trimmed and then covered over with a Portland cement plaster 1¼ to 2 ins. thick, which can be done only where there is no frost. This thin coating, when well laid, is permanent even on sandy or loamy banks.

Part II of the State Engineer's Report of California for 1888 gives an illustrated description, with cost, of several of these concrete linings.

Mr. Lesley.—In connection with what Mr. Parsons has said, drawing attention to the distinctions between brick and concrete lining for tunnels where trains are running through, without going into the actual cost, there are cases where large railroads are doing better in cost, or in practical results, by lining tunnels with concrete. One tunnel I have in mind is on the Philadelphia and Reading Railroad, at Perkasie, and another, the Musconetcong tunnel, on the Lehigh Valley, near Easton. The work was carried on very much as Mr. FitzGerald has carried on his. The arch in those tunnels had gradually come down with the water; I think in one there was 8 ins. of concrete; in the other, 6 ins. of concrete, the work being carried on continuously while the trains were running. These two tunnels were both lined with concrete, and have been in operation some two or three years.

Mr. Kenneth Allen, M. Am. Soc. C. E.—Some years ago, I believe, Mr. Chanute lined a number of culverts and arched bridges with Portland cement concrete. I think it was 6 ins. to 1 ft. thick, generally, and it stood very well.

Mr. FitzGerald.—If you will allow me a word, it seems to me the principal lesson to learn is, that if you are building a tunnel, it is cheaper and better to line it when you are building it than after it is in operation.

Mr. Brendlinger.—Referring to what Mr. Parsons has said, I cannot conceive how a concrete arch can be put in cheaper than a brick arch. Taking Mr. FitzGerald's report, it took him three seasons to do this work. I will warrant you that a good contractor will put that in in one season, with brick work.

There is another thing. Mr. FitzGerald says this cost $16 a yard,

his centers cost $1 400, and his shanties cost $1 200. A contractor would put that shanty up for $500 ; that is just the difference between an engineer doing the work and a contractor. If a contractor put in a brick arch—a good, conscientious contractor—he would do it for half the price.

Mr. Lesley made a statement about tunnel work ; he does not produce figures at all; he does not say what the cost of a brick tunnel would have been in that place, so you cannot make comparison there.

O. F. Nichols, M. Am. Soc. C. E.—I want to say a word only in answer particularly to Mr. Brendlinger's remarks. A great deal depends on the manner in which work is done. Concrete has been used recently in many ways that were not dreamed of before and with great success. Good Portland cement gives us superior concrete for strength, equal almost to rock itself ; but for best results it requires the utmost care, and a thorough and persistent inspection. I do not think there are many engineers who would entrust a piece of concrete work of this kind entirely to any contractor, no matter how good he might be. I have built a good deal of brick work, a great deal under contract and much by day's work. It is particularly difficult to make a good brick arch lining, even when you have the best conditions; the connections, and the binding at the key are difficult and there is some uncertainty about the action of the parts together when completed ; it requires a great deal of care to get the joints properly filled and the work all right. I think the work in this case could certainly be done cheaper in concrete than in brick. The great advantage, it seems to me, is that you obtain a more certain homogeneity in material in concrete and with greater ease than in brick work. The price seems to be remarkably low ; the work seems to have been done with the utmost care and attention and very economically, all things considered. The cost of this concrete lining could be compared more justly with concrete filling of pneumatic caisson than with concrete in open cutting, and such a comparison will show that the work was on the whole economically done. Of course, corporations like the city of Boston provide better shanties for their men than the average contractor does, even on the Croton Aqueduct.

Mr. FitzGerald.—I have had the pleasure of putting in a great many thousand yards of concrete in other places in the last two years, and that concrete generally costs between $5 50 and $7 a yard. It does seem rather a high price for Boston to pay for this concrete, $16 a yard; but take into consideration the circumstances, that you have got to provide the track, etc., etc., and then that the work cannot go on continuously, that it must be done at convenient times, and it must be done under exactly these conditions, that alters the whole thing. I was rather disappointed at the total cost of the concrete as a whole, but I believe the work was done as economically as

possible and very much better than a contractor could have done it. Of course a contractor can generally do work at lower prices than engineers, but I believe it would have cost a contractor nearly as much as $16 a yard. A contractor would not have been willing to have taken this work as it had to be done. He would have to hire his men, then discharge them again, and that would have added largely to the cost; it is like a contractor taking a piece of work that has not been tried, he would want a very large margin for contingencies.

John Bogart, M. Am. Soc. C. E.—I do not care to add to the quite full discussion of this evening, except, perhaps, to allude to one interesting remark of Mr. FitzGerald, that a lesson to be learned was, that if you have a tunnel through rock which is at all dangerous, you had better line it at the start. In a very large tunnel in the western part of this State, with the construction of which I was connected, we had not gotten very far before the character of the rock developed in such a way as to require a determination as to the desirability of lining. Although this increased the expense very considerably, a heavy lining of brick was put in.

There was to be in this tunnel a very rapid flow of water, the inclination being great, so as to take a very large amount of water through it, and it was determined that a brick which should be pretty nearly vitrified, burned very hard, would probably give the best surface to resist abrasion that we could economically secure, and some interesting experiments that have since been made upon various substances with a sand blast by a member of this Society, with whom I was associated in this work, have shown that the resistance of a very hard-burned brick to abrasion is about the greatest that occurs with any substance which could ordinarily be used in lining such a tunnel. This was the tunnel at Niagara Falls, built by the Cataract Construction Company. I hope before long to be able to present data on this subject in greater detail.

Mr. L. Luiggi.*—A gentleman desires to know the amount of water that is to be used for making a good Portland cement concrete mixture. I have here a report of a good many experiments made during 10 years in the harbor works of Genoa, where about 350 000 cu. yds. of concrete in concrete blocks were required, besides 100 000 cu. yds. of concrete laid under water, or more than 500 000 cu. yds. of concrete. These experiments were carried out with the greatest possible care. There are tabulated here the results of experiments, using different dimensions of sand, different dimensions of pebbles, broken stone, etc., the influences of the amount of water and a large number of the possible contingencies that happen in ordinary work. The conclusion is, that in a mixture of which about 70% of the volume is sand, 30% is water.

Then another point, speaking about the porosity of the concrete;

* Engineer in charge of Harbor Works, Leghorn, Italy.

there are here a good many experiments on the porosity of mortars and concrete. Porosity of mortar and concrete depends first of all on the visible voids between the grains of sand and shell and broken stone. If you take a part of sand and mix it with water in a glass, and stir it, you will see a certain amount of air bubbling up. There is an amount of water that you can hardly get out of the sand which makes a void that has to be taken into consideration. Then there is another void that is the result of the evaporation of water. These different voids have been separately calculated by experiment and the total void estimated. They made some blocks of mortar and concrete, and thoroughly dried and weighed them, and then immersed them in water for periods of two weeks, one month, etc. They were then weighed again, and evidently the difference in weight is the water that went into the voids.

A mixture is given that has only 5 per cent. of voids. The conclusion as to the porosity of mortars is, that mortars made with cement and coarse sand are the least porous of all mortars; and the most porous of all is one made with cement and very fine sand, and especially a mortar of only cement is more porous than with sand.

This porosity is not to be confounded with permeability.

The conclusion is, that with the same amount of cement a mortar made with coarse sand is more permeable than one made with fine sand; it is just the opposite with the porosity. With the same amount of sand, if you put less cement, the permeability increases.

Mr. Lesley.—The gentleman has stated that in determining the porosity of a block, that a block of concrete was put into water and the absorption was noted; was there any allowance made for the chemical combination between the cement and water making the hydrated double silicate of lime and alumina? In other words, a briquette of cement put in water at seven days, say, will show that the chemical combustion is only partly effected, moisture will show all over the broken section. It will be found, after from three to six months, that a little dry core will be gradually formed. At the end of four or five years moisture will only go into a very little section, the rest being dry and crystalline. That indicates that there is a chemical combination between the lime and water, taking up so much of the water. If the amount of water that the blocks absorb only was calculated, it might not alone determine that that block was so porous. It would have to be determined what percentage of water the cement would take up to form hydrated silicates of lime and alumina. Unless that were calculated, merely saturating a block would not indicate anything accurate or definite.

Mr. Luiggi.—Those experiments were made by immersing the blocks in water or in sand which was kept constantly under water two or three years; then they were dried in a warm room.

A Member.—What is the pamphlet in which these experiments are described?

Mr. Luiggi.—It is the "*Giornale del Genio Civile*," the number for October, 1893. I should say it would be worth while in this country to continue these experiments, because it is easier to take them than to make them anew.

Mr. FitzGerald.—There was one curious result noticed in mixing our sand and cement. If you take two casks of sand and one of cement and mix them, five times say, over and over again, they still fill the three casks, but the moment you put the water with them, the cement goes into the sand. It is almost impossible to mix the sand and cement together dry and reduce the bulk; the moment the water is added, the cement is driven into the voids of the sand, and of course the bulk is reduced.

Jos. P. Frizell, M. Am. Soc. C. E. (by letter).—The work of repairing the tunnel of the Sudbury Aqueduct, as detailed by Mr. FitzGerald, leaves little ground for criticism. The mixing of concrete in place I particularly approve of. Concrete cannot be transported and rehandled without sliding it, more or less, down inclines, which tends to separate it. The mortar sloughs down while the stones tend to roll, and the two materials usually land in different piles. A similar tendency to separation takes place when concrete rolls or slides down against a bulkhead or lagging. Concrete deposited against a flat surface, afterwards removed, is more likely to have cavities in the face than elsewhere in the mass of the work. A little consideration will show that this is unavoidable. Suppose concrete deposited on both sides of a thin plate. Every stone in contact with this plate bears against it at a single point. It could rarely happen that any two of these points are exactly opposite each other. On withdrawing the plate every stone will be free to advance a little, and will, on the whole, take a closer arrangement, which is the normal arrangement of the mass, and is, therefore, closer than the surface deposited against a wall.

In a work of this kind, when all operations are necessarily crowded, I could see no advantage in making two classes of concrete, one of American cement and one of Portland. The extra labor and loss of time incident to such an arrangement would more than counterbalance any saving in cost of cement.

From the leaving of the railroad track in place, making it, as it were, a permanent fixture of the tunnel, it is presumed that further falls of rock are apprehended. This suggests a question which Mr. FitzGerald's paper does not directly answer. If the falling had been allowed to go on and operations confined to the removal of débris from time to time, the fall of threatening portions being facilitated, what disaster was to be expected? Would this mode of dealing with the difficulty have brought the safety and permanence of the tunnel, as a

conduit, into question? As this question appears to have been decided in the affirmative, I infer that the rock was of a character liable to complete disintegration on exposure to the atmosphere.

Conceding that these repairs were carefully planned and skillfully executed, it is not to be forgotten that the construction of a tunnel which gives no occasion for repairs is a still better piece of engineering. In the excavation of a tunnel rocks will usually be disclosed whose characteristics are not familiar to the engineer. Some rocks, chiefly limestones, grow harder on exposure to the atmosphere. Some rocks crack, disintegrate and go to pieces under the same influence. No engineer can be presumed to have the geological and chemical skill to determine on inspection of a rock what will be its behavior on exposure. Nevertheless, this proposition, I think, cannot be di puted. A tendency on the part of a rock to disintegration can readily be detected by observing it for a few months under atmospheric influence, especially the action of frost. Samples of every variety of rock, whose weathering qualities are not familiar to the engineer, should be preserved in exposed situations and their behavior noted. On information so obtained, more confidently than on learned opinions of chemists and geologists, the engineer can determine the portions of the tunnel requiring to be lined.

Another point in the construction of conduit tunnels may be adverted to in this connection. In a lined tunnel considerable spaces often exist above the arch, which are usually filled solid with masonry, or, at least, supposed to be. This is required for two reasons: 1, to secure the arch against being lifted or burst up by internal pressure; 2, to avoid injury to the arch from falling rock. In a tunnel like the one in question, forming part of a gravity conduit with unbroken grade, the first consideration has no force. As to the second, the filling of these chambers with gravel or quarry spawls would be just as effective, large fragments being excluded. The first reason has a seeming application to a tunnel working under pressure, as a tunnel under the bed of a river, but even in that case its force is more apparent than real. Its only application is to the first filling of the tunnel, or to the very improbable case of its being pumped out. If the arch is tight, the natural seepage will insure the full external pressure on it at all times. I am told that the disused Washington tunnel discharges 1 000 000 galls. a day from natural seepage. If the arch is not tight, the filling of the tunnel will fill the spaces above the arch, and so equalize the pressure. The tunnel and lining can readily be arranged to secure this result. Experience at the Washington and New York tunnels shows how difficult it is to get these spaces filled with masonry. This part of the work is regarded as a legitimate subject of fraud, because the contractors and inspectors do not consider it essential. There would be no difficulty in getting these spaces filled with spawls

from the excavation, because that would be the easiest way of disposing of this material. The side walls should, of course, be carried up solid to the spring line of the arch. There would be no opportunity for fraud in this part of the work. A deficiency of 6 ins. or 1 ft. in the filling, which could not occur anywhere but at the top, would do harm, as a stone detaching itself from the roof and falling that distance upon a bed of loose material would bring no shock upon the arch.

On the other hand, the fraudulent execution of this work, as such frauds are usually practiced, viz., by running up bulkheads at intervals of 15 or 20 ft., leaving vacant chambers between, exposes the arch in these chambers to both forms of injury referred to, so far as the first is possible anywhere.

Foster Crowell, M. Am. Soc. C. E. (by letter).—Having been present at the meeting when this paper was discussed, but called away before having opportunity to speak, I desire to comment briefly on the paper, and particularly with regard to the point brought out in the discussion as to comparative cost of what have been designated as "contractor's" and "engineer's" work.

I cannot agree with some of the speakers either as to facts or conclusions. Mr. Brendlinger is correct in stating that brick work can be built for $8 per cubic yard, not taking any other values into account, but my experience does not justify me in supposing that any contractor would undertake to line an aqueduct of the dimensions given and in service for any such price. If Mr. FitzGerald were to itemize his figures and separate the extraordinary and unique portions of expense from the cost of the masonry, the concrete would be found to average about $14 per yard, for material and labor, and I doubt much whether my friend, Mr. Brendlinger, who is high authority on this topic, being always an excellent engineer, though sometimes a contractor, would undertake to do the work in the manner Mr. FitzGerald has described for much less, although the opportunity of selecting his own men would give him a great advantage over the officials of a municipal organization.

The reference to the Croton Aqueduct fails to add weight to the argument, excepting so far as it points to the difference in value between contractor's work and engineer's work. It is said that some contractors discovered just what that value was when they came subsequently to build the "difference."

It appears from the report of the Croton Aqueduct Commissioners of January, 1887, that the average contract price, on Sections 2 to 4, inclusive, of the new aqueduct (extending over 30 miles), for brick masonry lining made with Portland cement, was $12 37 per cubic yard; on three sections it was $13 50. Here the tunnel dimensions and volume of the work were much greater than in the work under discussion, and all the conditions vastly more favorable.

I would point out, also, that other things being the same, the application of concrete, from its nature, should be better and cheaper in a case of this kind than brick work, and the unavoidable inference is that, had brick work been used, the cost would have been greater. The cost of the various items, outside of the lining proper, was, doubtless, very much enhanced by reason of the local obstacles; for instance, the minor item of track appears to have amounted to about $1 for each cubic yard of concrete; this track was carefully and laboriously built, first distributing the rails along the entire length instead of delivering them from one end over the advancing track. The necessity of keeping the aqueduct in service was probably an adequate cause for this. I am inclined, from my own experience, to believe that a careful contractor would have declined to take this work, except on a labor and percentage basis, and that the result would have been no more favorable as to cost than Mr. FitzGerald obtained; while, without meaning to reflect on contractors, there would have been no such assurance of permanency as now appears. In regard to Mr. FitzGerald's experiments on proportions of ingredients of concrete, which he alludes to and which, I trust, he will give us the benefit of hereafter, I would point out, as I have already done in some previous discussions, that the question of proportion is one that can be best met by special gaugings with the material at hand; the size, porosity, fracture of the stone; the degree of fineness, shape and quality of the sand; the fineness and quality of the cement, even the chemical constitution of the water used, all come in. All contributions throwing light on these points are useful, but the physical characteristics of the materials must be given to make the experiments really valuable. In every case the voids in the stone, the voids in the sand, and the voids and chemical capacity for water of the cement, should be carefully measured; likewise the experiments should run through several proportions and kinds of water. Too much cement will weaken concrete just as too much glue weakens a cabinet-maker's joint. The proper fineness of the sand is of scarcely more importance than the shape of the grains, round grains giving maximum voids, minimum surface and greatest leverage on the cement, while a sharp sand reverses these conditions. Tests must be on the concrete and not on the cement.

James Duane, M. Am. Soc. C. E. (by letter).—There are a few points not fully brought out in Mr. FitzGerald's interesting paper and on which additional information seems desirable. *First.*—In view of the geological formation as described, why was not the tunnel lined as a part of the original construction; when, of course, the work could have been done to much better advantage, and there would have been less of it to do. *Second.*—While the author refers to the use of American cement concrete, and the table shows that 486 bbls. of American cement were used in the work, no description of the method of

making such concrete proportions of the different ingredients nor of the cost per cubic yard are given. It would be interesting if Mr. FitzGerald would kindly furnish this data. While employed as Superintendent of Construction in the 2d Light House District (coast of Massachusetts) some years ago, I constructed a number of heavy concrete foundations. Some of these were of Portland facing, Rosedale backing. In the light of the experience then gained I must agree with Mr. FitzGerald that the economy resulting from such composite construction was problematical, to say the least. As he well says, when you have a concrete gang well organized and everything working smoothly in all its details, a stoppage in any part disarranges the whole system. We soon found that it was generally more economical to increase the amount of ballast in our Portland cement concrete than to use Rosendale at all under ordinary conditions while the work went on harmoniously without any hitch. Another distinct advantage gained was in having a uniform binding material throughout the entire mass, thus securing uniformity in the time and method of setting, or of contraction, expansion, etc. It is believed that a more homogeneous structure was thereby obtained. It may be added that the difference of cost of the two kinds of cement was less to us than it would have been to an ordinary corporation, as the Government received its Portland cement duty free.

P. D. Ford, M. Am. Soc. C. E. (by letter).—In the discussion of the paper on "Lining a Water-Works Tunnel with Concrete," by Mr. FitzGerald, on the evening of February 7th, the lining of Perkasie Tunnel on the North Pennsylvania Division of the Philadelphia and Reading Railroad was mentioned.

In looking up some old memoranda on the subject made during the progress of the work, I find the following items which may be of interest in connection with the discussion of the paper of Mr. FitzGerald.

Perkasie Tunnel has a length of 2 117 ft., through a firm, compact rock, intersected transversely to the tunnel by several nearly vertical strata of rock of a seamy nature, varying in width from 20 to 125 ft.

From the time the first trouble was experienced from the falling of rock from these sections upon the floor of the tunnel, a gang of men was sent at regular intervals through the tunnel, making a careful examination and detaching every piece of loose rock and every piece of rock they could loosen. Notwithstanding this, the rock continued to loosen from natural causes, and the company determined to line one of the seamy sections with a béton of Portland cement, sand and fine gravel as an experiment. The most troublesome section, 125 ft. in length, was thus lined, before my connection with the road, in the year 1886, I think.

This proving a success, the lining of the remaining troublesome sections was undertaken in the fall of the year 1888. The inside di-

mensions of the lined portions of the tunnel were as follows: Vertical side walls from floor to springing lines, 14 ft.; chord between springing lines of intrados, 26 ft. 2 ins.; rise, 10 ft. 7 ins. The arch at key had a minimum thickness of 18 ins., and the side walls 12 ins.

Instead of using béton, as in the first section lined, the remaining sections were lined with a concrete composed of 1 part of Heyn's German Portland cement, 2 parts clean, sharp sand, and $3\frac{1}{2}$ parts of broken stone.

At the time the lining was done there was a single track laid on center line of tunnel, and in service. Traffic was unimpeded during the progress of the work. Four sections were lined, varying from 20 to 75 ft. in length. The work of furnishing, erecting and removing the centers was done by the railroad company. The concrete was furnished and laid by contract.

Void spaces of any considerable size above the extrados of the arch were partially filled with dry-stone work, forming a cushion on the arch about 3 ft. thick. Smaller spaces, say, up to 18 or 20 ins., were filled up solid with concrete.

The side walls and haunches of the arch were entirely of concrete. The extrados was inverted over the haunches, forming gutters which led to weep holes having iron pipes built in them, the pipes extending some 2 or 3 ins. inside the intrados for conveying water to the drainage ditches of the tunnel.

The dry filling was laid by the railroad company. The concrete was mixed on the surface of the ground, 34 ft. above crown of arch, and conveyed to the tunnel through a shaft from the bottom of which it was wheeled to the various sections.

The total amount of concrete laid in 164 ft. of arching above springing line, and 114 ft. of side walls, was 13 609 cu. ft., at 40 cents per cubic foot. I am unable to furnish the actual figures of cost of centering in place, but the total cost of the work, exclusive of dry filling, was estimated to amount to about 50 cents per cubic foot of concrete laid.

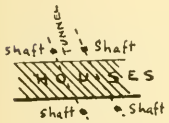
E. Sherman Gould, M. Am. Soc. C. E. (by letter).—A very useful paper, giving minute details of the execution of a very difficult undertaking. From its nature it does not afford much room for discussion, because no one would venture to criticise the methods employed without having a personal knowledge of the ground and the various conditions of the problem. Perhaps its most useful lesson is in showing the advisability of lining all aqueduct tunnels at once, before laying on the water, because they are pretty sure to need lining sooner or later.

Some interesting facts are incidentally brought out. The moot point of how many cubic feet go to a barrel (or "cask" as Mr. Fitz-Gerald calls it) of cement is settled by the statement that two barrels measure 6.84 cu. ft. Hence, we may take the barrel as cubing 3.40

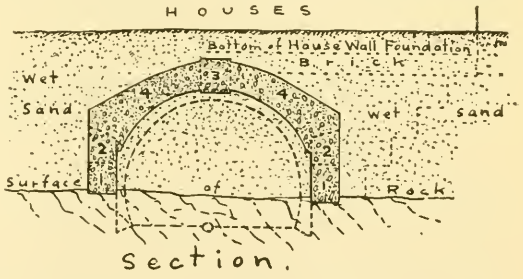
ft., for round numbers. I have recently been counting 10 barrels to the cubic meter, which agrees pretty well with the above.

Mr. FitzGerald states that the concrete was of exceptionally good quality. One would hardly have expected this from the manner in which it was mixed. I hope this point may elicit comment from other members, because if extra good concrete can be made in this way, then many of us have all along been taking a good deal of unnecessary trouble to accomplish the same result. As far as I have been able to figure it out from the data given, about 2½ cu. yds. of concrete were put into the work per day per man, all told, which is a high average.

E. E. R. Tratman, Assoc. M. Am. Soc. C. E. (by letter).—In a recent number of the *Proceedings* of the Institution of Civil Engineers (Vol. CXV, 1893–94, Part IV), is an account of a difficult piece of concrete underpinning on the Glasgow Underground Railway, a note of which may be of interest in connection with the projects for underground railways in New York. The tunnel was to pass close under a row of houses built on a stratum of fine sand saturated with water, overlaying a rock bottom. Four shafts were sunk to the rock, one at each end of each wall of the tunnel, as shown on the plan. Then the bottom headings



Plan.



Section.

(1) were driven and filled with concrete, and above these the headings (2), the upper surface of the concrete in these being sloped to form a springing. A crown heading (3) was then driven, and from this were built concrete ribs (4) until the two sides of the arch were continuous from end to end, after which the crown heading was packed with concrete. Under the house walls, wings of concrete were built and capped with brick built up tightly against the bottom of the foundation, as shown on the right of the section. When the concrete had set, the interior core was excavated and a brick lining built to the form shown by the heavy dotted line. The results were entirely satisfactory. It may be noted that this work was done by contract.

T. Kennard Thomson, Assoc. M. Am. Soc. C. E.—Would not an ordinary backing of earth or gravel have been good enough and much cheaper where the cavity was large ?

Additional information having been asked about the actual amount

of material required to make 1 cu. yd. of concrete, I would say that I had charge of some bridge foundations a few years ago, where the contractor used 1 cu. yd. of stone broken to pass a 3-in. ring; ½ cu. yd. of sand and ¼ cu. yd. of Rosendale cement to make 1.2 cu. yds. of concrete.

J. J. R. Croes, M. Am. Soc. C. E.—I understood one of the speakers to say that in the paper under discussion there was the first attempt that he had seen to give the exact proportions of stone, sand and cement in concrete. In Vol. III of the *Transactions* of the Society, published in 1874, there is a very full and detailed analysis of the proportions of sand and cement and stone, and also water, used in making the concrete for the Storage Reservoir Dam of the Croton Aqueduct Department at Boyd's Corners, N. Y., and a comparison of the same with concrete made for a reservoir of the St. Louis, Mo., Water Works, and a summary of the conclusions reached by the writer as to the best proportions of the materials for concrete.

Mr. FitzGerald.—In closing this discussion I desire to express my appreciation of the attention which this little paper has received. I feared that it would prove stupid, and should certainly never have read it had it not been for Mr. Brush. It shows that any work of which a careful record is kept may prove of interest to the profession.

There are several questions remaining to be answered. In regard to the relative cost of brick work and concrete it is safe to say that in Boston brick work costs double as much as concrete.

The ordinary price used in estimating on common work, as, for instance, in sewer work, where hard-burned bricks are used, is $12 per cubic yard for brick work laid in American cement mortar, and $14 if laid in Portland cement. It does not cost quite as much, for this includes the contractor's profit. In a piece of work recently constructed under contract, vigorously pushed and well handled, and containing 1 328 cu. yds., the cost, as ascertained by most careful force accounts, was as follows :

	Cost of brick work per cubic yard.
Labor	$2 89
Bricks	5 49
Sand	0 30
American cement	1 35
Centers	0 23
Miscellaneous	0 19
Total	$10 45

In this piece of work, selected from many others on account of its

general accuracy, 1.27 casks of cement were laid in each cubic yard of brick work. In Boston, Portland cement costs about $1 27 more per cask than American cement, the prices being $2 40, and $1 13 delivered.

In one cubic yard of brick work laid in cement there are from 560 to 580 bricks, without allowance for breakages, etc., and the cost delivered will average, say, $9 50 per 1 000. From the average of a number of force accounts it has been found that the labor costs $3 40 per cubic yard. We can now make another schedule as follows:

	Cost of brick work per cubic yard.
Labor	$3 40
Bricks	5 30
Cement	1 50
Sand	40
Centers	20
Miscellaneous	20
Total	$11 00

Where no large collar joints are present it is usual to reckon 1.23 casks of American cement to one cubic yard of brick work. If we use $11 for American cement brick work, we must use $12 50 * for same laid in Portland cement.

With the above figures I think it will be possible for any engineer to form his own opinion as to whether brick work would have been cheaper than concrete for the lining of the tunnel.

Mr. Crowell, I think, has shown very well that brick work in New York in tunnel has cost about $13 in large quantities, and under circumstances vastly more favorable than with an aqueduct in service.

To reassure Mr. Gould, I will repeat that the concrete turned out to be of most excellent quality. Mr. Gould's figures in regard to the quantity of concrete laid per man per day, however, need revising. I make it about 0.45 cu. yd. laid per man per day, instead of 2.5 cu. yds.

Mr. Duane asks as to the cost of the American cement concrete. This was used in the same proportions and cost $1 84 less per cubic yard, or $14 31. The usual prices in Boston are $6 and $8 per cubic yard for American and Portland cement, respectively. The reason the tunnel was not lined when first constructed was that, after consultation

* Generally $13, on account of its being a higher class of work, thinner sections, larger collar joints, etc.

with an expert geologist, it was thought unnecessary. This, I think, answers Mr. Duane's first question.

Mr. Frizell wishes to know what harm would have resulted from leaving the roof to fall, and my answer to this is, that aside from final stability of the roof, the falling rock might have killed some one, either during inspection or while cleaning the aqueduct. We clean twice a year.

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697.

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SPIRALS AND THEIR USE ON RAILROADS.

By A. S. C. WÜRTELE, M. Am. Soc. C. E.

READ MARCH 2D, 1894.

The constantly increasing rate of speed at which railroad trains are now run calls for a corresponding increase of care on the part of the constructing engineer in laying out and improving lines of railroad.

It is no longer sufficient simply to counteract the centrifugal force on curves by elevating the outer rail according to the well-known rules. This was well enough for a speed of, say, 30 miles per hour, but now we look forward to a speed of 100 miles per hour, and the smooth running of a train at such speed requires a more complete system of rail elevation than has been in use, for the reason that at the point of curve the rail cannot be raised suddenly to its proper elevation, but only gradually, so that, at entry on a curve, a shock occurs from lack of compensation for the centrifugal force suddenly developed. To obviate this shock, systems of spirals at the commencement of curves have been adopted and are now being used, so that, from the gradually diminishing radius of curvature of the spiral, the outer rail can now be put at its proper elevation from the very point of a curve.

Twenty-five years ago, on location, the writer compounded all sharp curves by putting in 100 ft. of a 1° curve at each end, and no doubt with good result for those early days of the development of the railroad system as it now exists.

In Great Britain, Mr. Gravatt, about 1829, advocated the use of the curve of sines instead of the circular curve, and about 1842 W. Froude invented the "curve of adjustment," to be used at each end of a circular curve. The length of this curve of adjustment was 300 times the proper elevation of the outer rail for greatest speed. The circular curve was set in towards the center by a small distance, found by dividing the square of the above length by 24 times the radius. One-half the length was set off each way from the point of curve, and the curve of adjustment was finally set out by offsets from the curve as set in towards the center, the offsets being proportional to the cubes of distances from the ends, the cube of half the length, and half the distance set in.

In all flattening of ends of curves the main curve must necessarily be slightly sharpened, and this is effected by setting the curve in towards the center.

Wellington in 1881 and Searles in 1890 have dealt exhaustively with the subject of railroad spirals;* but as the thing has not been practically used in general, and is probably little studied or understood by many working engineers, the writer is emboldened to present the following general view of spirals, and to add a system he has worked out for the use of a single spiral, uniform for all curves, by using different lengths of spiral, as indicated in the accompanying table.

The Spiral Generally.—The equations of spirals are of two forms—1st, the radius vector a function of the angle turned in circular measure, and, 2d, the perpendicular on the tangent from the pole a function of the radius vector: so that there are an infinite number of spirals, of which some of the best known are indicated.

One large family of spirals has the equation $r = a\,\theta^n$, in which every change of n gives a different spiral.

* Spiral Literature:

1881. Wellington in *Railroad Gazette.*
1884. Man in *Transactions,* Am. Soc. C. E.
1890. Searles in book form.
1891. Gribble " " "
1892. Caine in *Transactions,* Am. Soc. C. E
1892. Ward " " "
1893. Crandall in book form.

$n = 1$ gives $r = a\,\theta$ or $p = \frac{r^2}{\sqrt{a^2 + r^2}}$ the spiral of Archimedes;

$n = 2$ gives $r = a\,\theta^2$ the parabolic spiral;

$n = -1$ gives $r = \frac{a}{\theta}$ or $p = \frac{a\,r}{\sqrt{a^2 + r^2}}$ the hyperbolic spiral;

$n = -\frac{1}{2}$ gives $r = \frac{a}{\theta^{\frac{1}{2}}}$ the lituous spiral;

$n = \frac{1}{2}$ gives $r = a\,\theta^{\frac{1}{2}}$

which may be called the railroad spiral, as it has properties rendering it easy to use, and it gives good results. Some of these properties are as follows: The angle of the tangent with the radius vector is double the angle of the radius vector with the origin line; the total angle turned to any radius of curvature is equal to the angle turned in a curve of the same radius in a distance equal to half the radius vector; the distance the spiral will fall inside the curve is one-fourth the ordinate at the end of the spiral.

The helicoidal parabolic spiral has the equation $(a - r)^2 = 2\,a\,b\,\theta$. The logarithmic spiral is given by $r = a^\theta$ or $\log. r = \theta$; this is also called the equiangular spiral as the angle between the tangent and the radius vector is constant. Had the force of gravity varied as the cube of their distance, the planets would have continually receded from the sun in a logarithmic spiral, as shown by Newton.

Cotes' spirals are of interest as being the orbits in which bodies, projected with different velocities in different directions, would be held by an attracting force inversely as the cubes of the distances.

The equations are $p = \frac{b\,r}{\sqrt{a^2 + r^2}}$

$b = a$ gives a log. spiral, and $p = \frac{b\,r}{a}$ gives $\theta = \frac{b}{\sqrt{a^2 - b^2}} \log. \frac{r}{a}$

$a < b$ gives $\theta = \frac{b}{c} \log. \left(\frac{\sqrt{a^2 + c^2} - c}{r}\right)$ where $c = \sqrt{a^2 - b^2}$

$a > b$ gives $\theta = \frac{b}{c} \sec. \frac{^{-1}r}{c}$, or $r = c \sec. \frac{c\theta}{b}$ where $c = \sqrt{b^2 - a^2}$

Differential Equations.—The general differential equations necessary in solving spiral calculations are:

$$\frac{dA}{d\theta} = \frac{r^2}{2}. \qquad \frac{ds}{d\theta} = \sqrt{r^2 + \frac{dr^2}{d\theta^2}}$$

$$\frac{ds}{dr} = \frac{r}{\sqrt{r^2 - p^2}} \qquad \frac{dr}{dp} = \frac{R}{r} \qquad \frac{d\theta}{dr} = \frac{p}{r\sqrt{r^2 - p^2}} = \frac{t}{r^2}$$

$$\frac{1}{p^2} = \frac{1}{r^2} + \frac{1}{r^4}\frac{dr^2}{d\theta^2}$$

In all the preceding formulas, r is the radius vector, θ is the circular measure turned by r, p is the perpendicular from the pole on the tangent, s is the length of arc from the pole, t is the subtangent from the pole to the tangent and at right angles to r, A is the area between the radius vector and spiral.

From the given equation of a spiral the differentials are easily obtained; and, by insertion in the differential equations, and by integration between desired limits, the spiral is reduced in all its conditions.

The spiral $r = a\,\theta^{\frac{1}{2}}$ is reduced as follows:

$$\theta = \frac{r^2}{a^2},\ p = \frac{2\,r^3}{\sqrt{4\,r^4 + a^4}} = \frac{2\,r^3}{a^2} \text{ nearly};$$

r^4 being very small compared to a^4.

$$R = \frac{(4\,r^4 + a^4)^{\frac{3}{2}}}{6\,r\,a^4 + 40\,r^5} = \frac{a^2 + 3\,a}{6\,r} \text{ nearly};$$

$$\text{Sin. } \beta = \frac{p}{r} = 2\,\theta = \frac{2\,r^2}{a^2} \text{ nearly};$$

$$x = \frac{r^3}{a^2} = r \text{ sin. } \theta = \frac{a^4}{216\,R^2} \text{ nearly};\ d = \frac{x}{4}.$$

$$A = \frac{a^2}{2} \times \frac{\theta^2}{2} \qquad A = \frac{r^4}{4\,a^2}$$

Where R is the radius of curvature at the end of r, x is the ordinate at the end of r, d is the distance the end of r falls inside the circular curve, β is the angle between the tangent and r, x is proportional to the cube of r.

Determination of a.—Take a 3° curve, and, at 100 ft. on the curve, a radius vector from 50 ft. back of the point of curve will give the tangent to the spiral and to the 3° curve parallel, as $I = 3\,\theta = 2\,v$, I being angle of intersection and v deflection angle. From this $a = \frac{r}{\theta^{\frac{1}{2}}} = \frac{150}{(.0175)^{\frac{1}{2}}}$ = 1134, where .0175 is circular measure for 1°.

The tangents are now parallel, but the radius of curvature of the spiral is not equal to the radius of the 3° curve, but

$$r = \frac{a^2 + 3n}{6\,R} = 112.2.$$

When the radius of curvature of the spiral is 1 910, the same as the curve taken, and $\theta = \frac{r^2}{a^2} = .00979$, which multiplied by 57.296° reduces the circular measure to 33′.5, and consequently β = 67′, or the total angle turned is 101′. Now this angle will be turned by the 3° curve in 56.1 ft., or half of r, so that, starting the spiral 56.1 ft.

back from the point of the curve, and running $r = 112.2$, at angle of 33'.5 the tangents will be parallel and the radius of curvature of the spiral at the end of r the same as radius of 3° curve. But the points will not be together, the spiral falling inside the 3° curve by the amount $d = \frac{r^3}{4a^2} = 0'.27$, and by setting in the curve by 0.27, a complete curve will be formed in continuation of the spiral.

The table of the spiral $r = a\theta$, calculated for every 20 ft. of r, with a equal to 1 134, as above deduced, is suitable for railroad curves from 2° to 6°, and it can be set out by the offsets x with r; or by transit angles $\Theta°$ with r, when angle β gives the means of turning on the tangent at the end of the spiral.

The second table fixes the length of spiral for the curves it includes, and is to be used with the first table, which indicates the end of the spiral as well as the commencement at — $P\ C$, back from point of curve, also the final offset x, and distance $\frac{x}{4}$, for setting in the main curve.

Very sharp curves, as for street railways, of 40 to 100 feet radius, require correspondingly short spirals; and the third and the fourth table of spiral $r = a\,\theta^{\frac{1}{2}}$, calculated for every 5 ft. of r, with a equal to 58.1, obtained in a similar manner to the above, is suitable for curves from 30 to 60 ft. radius.

In case the offset of curve is fixed or another offset desired than as given in the table, the proper a for the required spiral will be had from the equation $d = \frac{a^4}{4 \times 216\,R^3}$, and r from $r = \frac{a^2 + 3\,a}{6\,R}$; when the angles for any length of r can be had from the general equation $\theta = \frac{r^2}{a^2}$ and the offsets x by $x = r$ sin. θ, or by $x = \frac{r^3}{a^2}$ and the spiral begins at $\frac{r}{2}$ from point of curve.

The above reductions have been made from the approximate equations of p and R, which are sufficiently correct for all practical purposes in the part of spiral used.

There is, of course, nothing new in the mathematical reductions of spirals, but only in their application to practical purposes. The writer consulted principally Hall's "Integral Calculus," "Encyclopædia Britannica," and Davies & Peck, and has endeavored to collect together some widely scattered information on the subject of spirals, and to indicate the practical application of the same.

Spiral for Railroad Curves.

I.— $r = a\,\theta^{\frac{1}{2}}$ $a = 1\,134$

r	$\theta°$	$\beta°$	x ft.	y ft.	R ft.
20	1′.07	2′.14	.006	20	10 714.3
40	4′.28	8′.56	.05	40	5 357.2
60	9′.63	19′.26	.16	60	3 574.8
80	17′.11	34′.22	.40	80	2 678.6
100	26′.74	53′.48	.78	100	2 142.8
120	38′.50	1° 17′.00	1.34	120	1 787.4
140	52′.41	1° 40′.82	2.13	140	1 530.6
160	1° 8′.43	2° 16′.86	3.18	160	1 339.3
180	1° 26′.63	2° 53′.26	4.54	180	1 191.1
200	1° 47′.00	3° 34′.00	6.22	199.9	1 071.4

II.—

r	θ	β	x ft.	y ft.	R ft.	c	d ft.	$-P\,C$ ft.
74.8	15′.0	30′.	.33	74.8	2 865	2°	.09	37.4
93.5	23′.4	46′.8	.64	93.5	2 292	3° 30′	.16	46.7
112.2	33′.5	1° 7′.0	1.09	112.2	1 910	3°	.27	56.1
130.9	45′.8	1° 31′.6	1.74	130.9	1 637	3° 30′	.44	65.5
149.6	59′.8	1° 59′.6	2.60	149.6	1 433	4°	.65	74.8
168.3	1° 15′.7	2° 31′.4	3.66	168.3	1 274	4° 30′	.92	84.1
187.0	1° 33′.5	3° 7′.0	5.09	187.0	1 146	5°	1.27	93.5
224.9	2° 14′.8	4° 29′.6	8.82	224.7	955	6°	2.21	112.5

Spiral for Street Railway.

III.— $r = a\,\theta^{\frac{1}{2}}$ $a = 58.1.$

r	$\theta°$	$\beta°$	x ft.	y ft.	R ft.
5	25′	50′	.04	5.00	118.3
10	1° 42′	3° 24′	.30	10.00	59.1
15	3° 49′	7° 38′	1.00	14.97	39.4
20	6° 47′	13° 34′	2.36	19.86	29.6
25	10° 37′	21° 14′	4.61	24.57	23.7
30	15° 17′	30° 34′	7.91	28.94	19.7

IV.—

r	$\theta°$	$\beta°$	x ft.	y ft.	R ft.	d ft.	$-P\,C$ ft.
8.46	1° 13′	2° 26′	.18	8.45	70	.045	4.23
9.86	1° 39′	3° 18′	.28	9.84	60	.07	4.93
11.84	2° 22′	4° 44′	.48	11.82	50	.12	5.92
13.14	2° 56′	5° 52′	.67	13.13	45	.17	6.57
14.78	3° 43′	7° 26′	.96	14.76	40	.24	7.39
16.90	4° 51′	9° 42′	1.43	16.84	35	.36	8.45
19.72	6° 36′	13° 12′	2.27	19.59	30	.57	9.86
23.66	9° 30′	19° 00′	3.90	23.34	25	.975	11.83

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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698.

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THE ELECTRIC STATION OF THE CITIZENS' LIGHT AND POWER COMPANY OF ROCHESTER, N. Y.

By ROBERT CARTWRIGHT, M. Am. Soc. C. E.

READ MARCH 7TH, 1894.

WITH DISCUSSION.

The inventions and improvements in electrical science at the present day are so rapid and comprehensive that a very little time makes great changes in the development of the machinery and the application of electricity to many new channels of industry. Already has the small steam engine been relegated to "innocuous desuetude" by this unknown but potent agent, because of the readier application, cleanliness and economy of the latter. The day cannot be long delayed in this progressive age when the generation of electricity by hydraulic power and its conservation in a transportable form will effect a revolution in many of its present objectionable features, and we will see the expensive copper wire at least restricted in its use, if not entirely displaced by storage, in a practical and economical manner. Do we not have energy of almost infinite capacity concentrated in the form of gunpowder, dynamite, nitro-glycerine and many other kindred compounds, and is it expecting too much of the inventive genius of this in-

ventive age to confine this subtle power, so that we can release and utilize it at will? "'Tis a consummation devoutly to be wished" that some Edison or Tesla, some Thomson or Julien, will produce it from the combustion of coal, without the intervention of engine or boiler, water-wheel or dynamo, as an intermediate part of the plant. What electricity is, we know not. Its effects and its affinities comprise the sum total of our knowledge. Its development, instead of being the result of studied theoretical laws, is almost entirely experimental. The telephone and phonograph, the dynamo and transformer, are all the products of experimental research. Their inventors did not know what they sought, until, like Pandora opening her box, they released a series of perfected, though previously unknown, results.

The consolidation of the three gas companies and the three electric companies of Rochester, N. Y., into one organization, known as the Rochester Gas and Electric Company, was effected in 1892. This combination seems to have awakened in the mind of some of Rochester's citizens the old saying that "competition is the soul of trade," and hence the birth of the Citizens' Light and Power Company. Some of the stockholders in the different companies sold out their interests when the consolidation was entered into, and organized an independent company, whose plant, designed and constructed by the writer, is in operation, supplying electricity for light and power. Its equipment is of the latest and most improved machinery under the Westinghouse system. As the work embraces some interesting features, a description of the power plant in detail is furnished.

The city of Rochester, N. Y., is almost evenly divided by the Genesee River running through it from south to north, and having within the corporate limits of the city a total fall of about 257 ft., in a succession of three falls and rapids. The water is used over four times before it reaches the level of Lake Ontario. In former years when the drainage area of the country supplying the Genesee with water was covered with forests, the flow was constant and reliable, but to-day an advanced civilization has cleared off the land, and the consequence is that a dependence on the river, as a means of supplying power, is very irregular and uncertain, almost realizing the old adage of "a feast to-day and a famine to-morrow." This has become so unsatisfactory that the city's interests are jeopardized by it, and measures are now being taken to store the water, at a point near Mount Morris,

by damming it back in the ravine to the south. A bill is now being considered in the State Legislature, looking to this end. As the State of New York has the first right to the water of the Genesee, or so much thereof as is needed for feeding the Erie Canal, at times of drought in the summer the volume reaching the city is sometimes only 4 000 cu. ft. per minute, as per the gauging records of E. Kuichling, M. Am. Soc. C. E., and Chief Engineer of the water works. This, divided among the many industries dependent upon it, becomes practically useless. By impounding the water, as stated, the amount confined from melting snows and heavy rains would be dispensed at Rochester, so that a constant power of 30 000 H. P. per day would be realized, instead of passing through the city at times of freshets entirely unused.

In the opinion of the writer there is no other public improvement that will inure to the benefit of Rochester so much as the storage of the water of the Genesee. Parks and boulevards may easily wait until the utilization and application of all of the water of the river will so enhance the position of Rochester, as a city of reliable and cheap water power, that manufacturing capital will find a desirable investment in it, and fill the entire ravine below the falls with hives of industry which in a very short time would enable the city to adorn itself without the imposition of onerous taxation.

The fact of water storage in the near future being a possibility has been considered by the writer in designing the power station, so that advantage may be taken of it, when realized, without unnecessary change or expense. By reference to the accompanying plates, the location, disposition and construction of the work will be readily understood.

In August, 1892, the Citizens' Light and Power Company acquired the property now occupied by its station on Brown's Race, at the foot of Factory Street. Nothing but a mass of ruined walls and débris gave evidence of its having been previously occupied. The site was that where once stood the Jefferson and Clinton Flouring Mills, which were destroyed by fire under such peculiar circumstances that the writer may be pardoned for a narration of them.

Fig. 1 shows a plan and sectional elevation of the property at the time of its purchase by the Citizens' Light and Power Company. As will be seen, a main sewer traverses the whole length

of the Jefferson lot from Mill Street to the Genesee River. The sewer was 6 x 6 ft. in size, and was cut under Brown's Race by tunnelling the rock some 20 ft. below the race bottom, the lower portion built on the rock slope and covered with earth. On December 21st, 1887, the Municipal Gas Company, whose property was on the line of said sewer about 3 500 ft. distant, was being supplied with naphtha by the Vacuum Oil Company through an underground pipe connecting the two properties. The naphtha was forced by pumps to a tank at the gas works. A short time prior to the delivery of the last large supply of naphtha, the connecting pipe was broken at an undiscovered point in the vicinity of a rock excavation adjacent to the line of the aforesaid main sewer, and about 15 000 galls. of the naphtha was pumped into the sewer instead of being delivered into the tank. The naphtha was carried along on the sewage until the whole length of the sewer to the outlet at the river was charged with its vapor. By some means fire was communicated to it through an untrapped lateral sewer, and an explosion occurred that blew up the Jefferson Mill, as well as many of the manholes along the line of the sewer, the flames at every point of issue rushing out to a height of 30 to 40 ft. The resulting fire destroyed the Jefferson Mill, as well as the adjoining Hinds' Mill, leaving them in ruins, as well as causing the loss of five human lives. The Clinton Mill had been burned down some time previous. The Hinds' Mill was rebuilt in 1888, but the others were as left by the fires when the Citizens' Light and Power Company purchased the property. This was the condition of things when the writer was called on to plan an electric-light station.

The Jefferson Mill had two water wheels located near *A*, Fig. 1, operating under a collective head of 56 ft. The discharge from these wheels, as well as that from the Hinds' Mill, ran to the pond *B*, which had a dam at the west side of Falls Street. The water from this pond was used on the two wheels *C* and *D*, with a head of about 25 ft., power from these lower wheels being transmitted to the Hinds' and Jefferson mills by means of wire rope.

After due consideration of all the essential features, the plan as shown on Fig. 2 was decided upon, the whole station being placed below the site of the mills upon what was considered almost useless land. As the race front was the portion that gave value to the property, the Company possesses the same race frontage intact, while

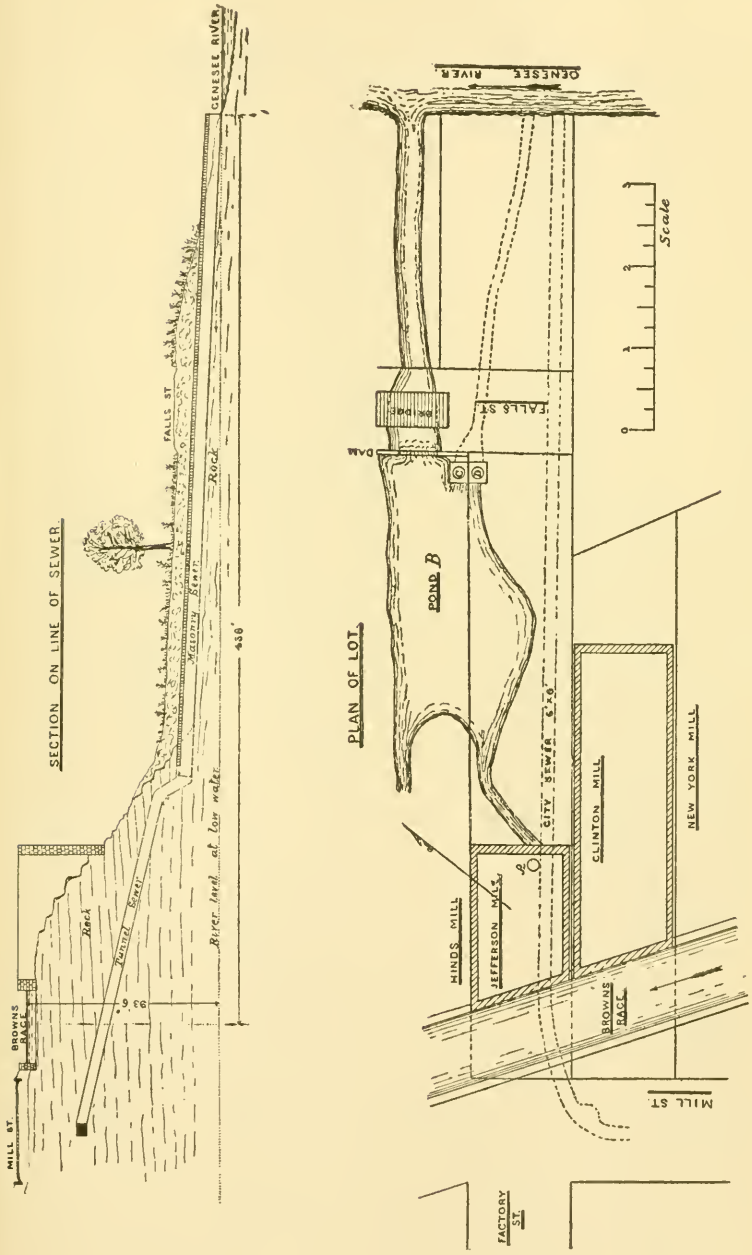

SECTION ON LINE OF SEWER.
GENESEE RIVER
FALLS ST
Rock
Masonry Sewer
River level at low water
438'
93.6'
Rock
Tunnel Sewer
BROWNS RACE
MILL ST.
PLAN OF LOT.
GENESEE RIVER
Scale
0
1
2
3
FALLS ST
DAM
POND B
CITY SEWER 6'x6'
CLINTON MILL
NEW YORK MILL
HINDS MILL
JEFFERSON MILL
BROWNS RACE
MILL ST.
FACTORY ST.

FIG. 1.

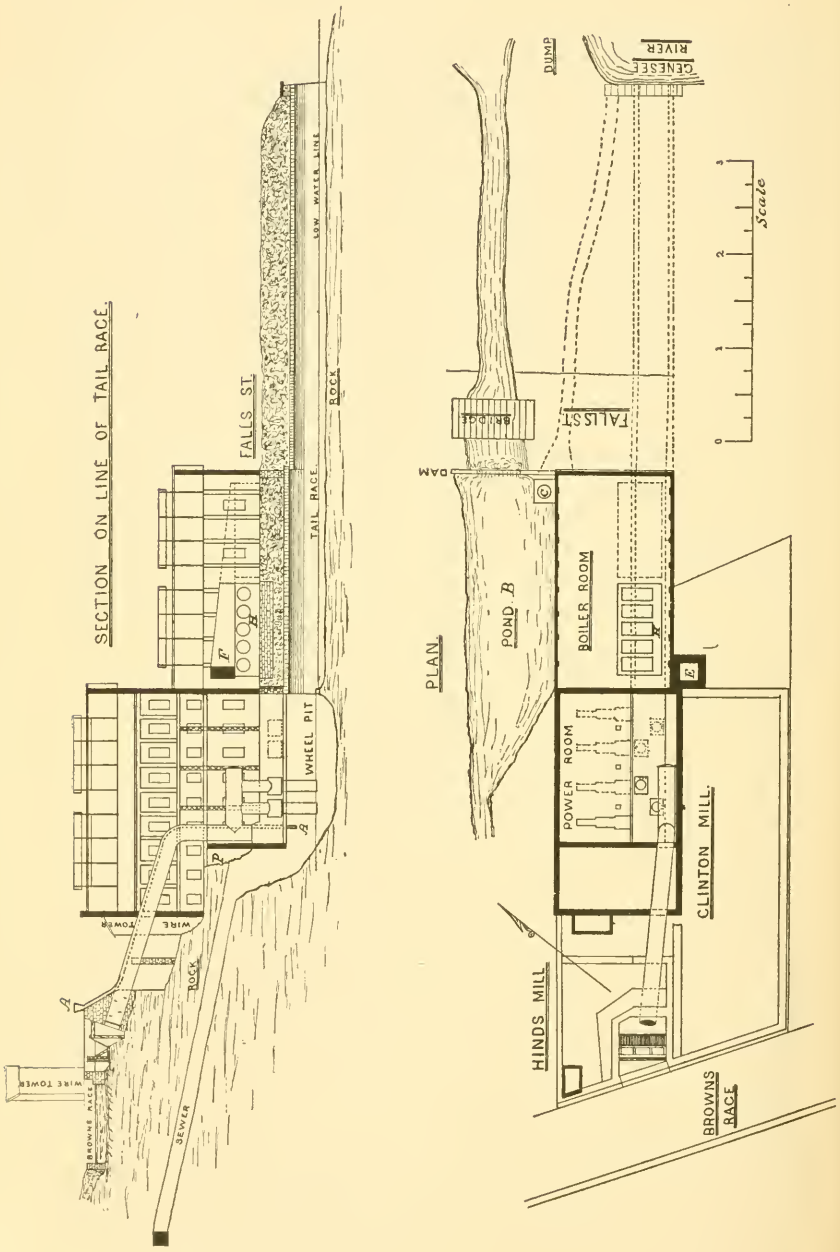

SECTION ON LINE OF TAIL RACE.
FALLS ST.
LOW WATER LINE
ROCK
TAIL RACE
WHEEL PIT
WIRE TOWER
ROCK
SEWER
BROWNS RACE
WIRE TOWER
PLAN.
DUMP
GENESEE RIVER
BRIDGE
FALLS ST.
DAM
POND B
BOILER ROOM
POWER ROOM
CLINTON MILL.
HINDS MILL
BROWNS RACE
Scale
0
1
2
3

FIG. 2.

it has acquired a complete power station besides. Work was commenced on the 28th of November, 1892. As the discharge from the sewer was continuous, and had to be maintained while the work was being prosecuted, the first step taken was to direct the sewage water into the pond *B*. This was successfully done by a large wooden trough placed at such a point in the sewer as would pass the water by gravity into the pond *B*. A coffer-dam of sufficient dimensions and height was placed at the river end, so as to shut out the river water in its various fluctuations. Steam pumps kept the water down, so that the rock at the lower end was bared, and upon this the masonry of the side walls of the tail race was commenced, and carried up to 6 ft. above low water-mark of the river. The race was made 13 ft. wide in the clear. An arch of 7½ ft., internal radius was thrown across the top. The rock was excavated in the center of the race 2 ft. below the low water-mark, which was considered the datum point of the whole work. This construction was carried out in open cut (the old sewer being removed as the work progressed) to a point coinciding with the west side of Falls Street, a distance of 165 ft. from the river face of the tail race. Here a change was made, making the arch semi-circular, with an internal radius of 6½ ft. This was done in order to better support the superincumbent weight of steam boilers that were placed directly over the tail race, as shown on Fig. 2. These dimensions were maintained for a distance of 95 ft. west of Falls Street, making a tail race proper 260 ft. long. At this point the wheel pit commences. The side walls of the race are built 2 ft. thick, and a skew-back placed 6 ft. above the datum line to receive the arch, which was also 2 ft. thick. The face of tail race masonry at the river end is rock-faced, and is carried up above the top of the arch and covered with point-dressed coping stone. At the inner end of the tail race, adjoining the wheel pit, is placed a level sill or spill, whose top is 1 ft. above the datum, to maintain a seal of the draft tubes of the water wheels. The wheel pit was enlarged by under-cutting the rock, as shown by Fig. 3, and deepened, so that its bottom is 6 ft. below the bottom of the draft tubes. This gives free discharge to the water from the wheels and prevents any reaction. The pit is 64 ft. long, and on the wheel floor is 14 ft. wide in the clear. The wheels are supported on double 16-in. steel I beams, resting 2 ft. in the rock at the ends, in recesses cut therein. Above the wheel floor the pit is lined with 12. ins of brick work on the sides. The ends of the I beams

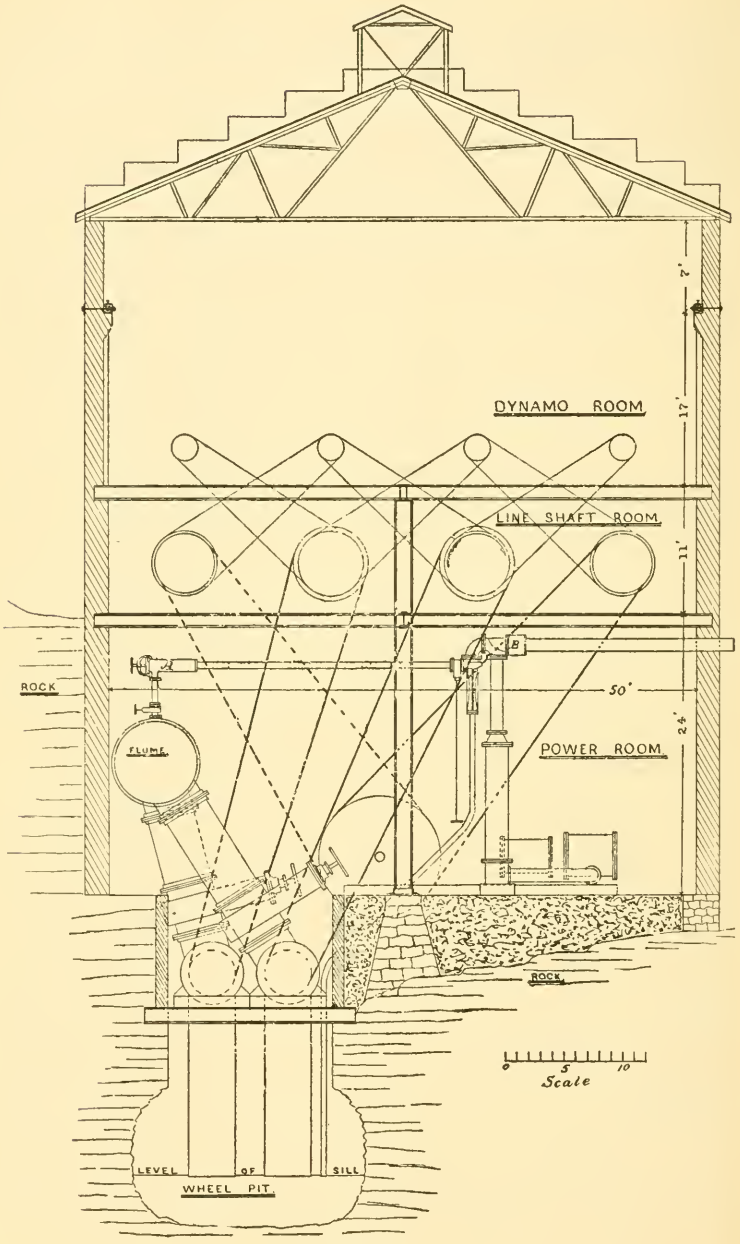

DYNAMO ROOM
LINE SHAFT ROOM
POWER ROOM
ROCK
FLUME
B
50'
7'
17'
11'
24'
ROCK
0 5 10
Scale
LEVEL OF SILL
WHEEL PIT

FIG. 3.

are solidly built up with brick, filling the recesses. The wheel floor is laid with 2-in. matched and surfaced hemlock plank. Upon this at right angles to the plank is laid 1½-in. planed and matched hard maple flooring, with a layer of asbestos flooring paper between the two. This was done in order to cut off any possible odor arising from the sewer. The wheel pit floor is 15 ft. 3 ins., and the power-room and boiler-room floor 25 ft. above the datum line.

By the 8th of March, 1893, the tail race, wheel pit and the foundation of boiler house were completed, and the sewage was turned into the wheel pit. All of this masonry was laid in American Portland cement, of the Columbia brand. The work was prosecuted continuously, excepting Sundays, during the whole of the cold winter of 1892-93, the temperature being as low as —6° Fahr. at times. But one half day was lost during the whole time, when a sleety rain that froze as it fell rendered it impossible for the men to retain a foothold on the work. All the mortar was salted, to prevent freezing. The foundation of the buildings and chimney are all upon the rock, with the exception of a short space where the water wheel D, Fig. 1, had been removed.

On March 16th the brick work of buildings and chimney was commenced. The walls of the boiler house, as well as the chimney, are laid in quick-lime mortar, but all the rest of the whole work is laid in cement mortar. For the main building the walls were laid as follows:

An inside and outside course of wall was laid two bricks high, then the center of the wall was filled with grout and the bricks placed in the grout, the bricks having been previously wetted. Headers were introduced every seven courses, and alternated on the inside and outside of the walls. All bricks were hard-burned sewer bricks, and joints were struck on the inside and outside of the walls. When completed, the inside faces of all the rooms were painted with white Aquol paint. The walls of the boiler house are 12 ins. thick, and reinforced on the inside by 4-in. pilasters, 16 ins. wide, placed immediately under the roof trusses, spaced 10 ft. 4. ins. apart, centers. The walls are 24 ft. high to the eaves, with a gable on Falls Street.

The walls of the main building are 59 ft. high from foundation to eaves, divided into a power-room 24 ft. high, and a line shaft-room 11 ft. high. To this height the walls are 2 ft. thick. At the floor of the dynamo-room the wall is reduced to 20 ins. thick, and reinforced on

the inside with 4-in. pilasters 20 ins. wide, spaced 10 ft. 4 ins. apart, centers. At a point 17 ft. above the floor the brick are corbelled out, joining the pilaster heads and forming a bearing for a line of 8 x 8-in. white oak stringers, which are bolted through the walls, to carry an overhead traveling crane. Above this the walls are carried up to the eaves, 20 ins. thick.

The buildings are covered with steel-trussed roofs, having a rise of 13 ft. in the center. Principals are spaced 10 ft. 4 ins. apart, centers. Span of dynamo-room, 50 ft.; and of boiler-room, 48 ft. Each roof has two ventilators, for light and air, which are about 30 ft. long each. Upon the trusses steel purlins of 6-in. channel beams are fastened on edge. To the web of these purlins are bolted 2 x 6-in. stringers. To these are nailed the roof boards of $1\frac{1}{8}$-in. flooring, and the whole covered with sanded tar roofing paper.

As the sides of the Jefferson lot are not parallel, being 50 ft. wide at Falls Street and only 48 ft. 9 ins. wide at a point coinciding with the west end of the main building, an agreement was made with the Hinds' Mill Company to place so much of the building upon their property as would make the buildings parallel. As a consideration therefor a party-wall interest in the north wall of main building was given them. By this arrangement the roof trusses were uniform, and the result was a considerable saving in the cost of roof construction.

A chimney is located as shown at E, Fig. 2, into which are carried, by the smoke jack F, the smoke and gases from the batteries of boilers. The chimney foundations are on the rock. It is 14 ft. square at the base, and is carried up 125 ft. high, with a straight flue 8 x 8 ft.

As soon as the season was advanced far enough to preclude freezing, the space in the boiler-room over the tail race was filled and solidly rammed down to a depth of 6 ft. below level of floor, and a solid foundation of rubble work, laid in cement mortar, was built to receive the boiler setting for the first battery of five boilers, as shown by Fig. 2.

On Sunday, May 7th, an accident occurred that resulted in the carrying away of the west wall of the power-room. A heavy freshet was running in the river, and the man having charge of the inlet gates to Brown's Race, by some miscalculation, allowed so much water to enter that it overflowed the old race wall at the company's property, and ran down and filled the pocket P, Fig 2, and, being confined between the rock and the wall, the hydrostatic pressure was great enough to

throw the wall in large masses into the power-room and wheel pit. The wall was 50 ft. long by 23 ft. high by 2 ft. thick. It started at the bottom, as shown by the position in which it lay, actually tearing out the rock which was still clinging to the brick work at the bottom.

The old race wall was very defective and was replaced with a new wall, the whole length of the front of the property being 103 ft. 6 ins. As the water in the race could not be let out excepting on Sundays between the hours of 8 A. M. and 5 P. M., it was necessary to put in a coffer-dam while the wall was built and the head gates put in place. All the material was made ready for a speedy placing in position of the dam, and on Sunday, May 28th, it was successfully accomplished in the allotted time. The bottom of the race was rock, rather irregular on top, and the use of flour sacks partly filled with dry cement mortar was resorted to, in order to fill the gaps under the sheet piling, a layer of bags being placed at the bottom and tamped down so as to pack closely. The cement absorbed the water and set as a solid stone, filling every crevice, the interior being filled with 18 ins. of clay puddle, also rammed down. In order to prevent the dam from slipping on the rock, holes were drilled in the rock about 6 ft. apart along the line of the race wall, and 1¼-in. round iron pins dropped into them. A hemlock stringer 6 × 6 ins. was laid against them on the rock and the first course of sheet piling nailed to it. The girts above were 6 × 6 ins., and were stayed with ⅝-in. bolts and cast-iron washers, to prevent the puddle from spreading the dam. The first course of sheet piling on each side was 1½ ins. thick, and another course of 1 in. was fastened over, to batten the joint. Fig. 4 shows a section of the coffer-dam. The old race wall was removed in sections and replaced by the new wall; the rock was removed and part of the forebay with the

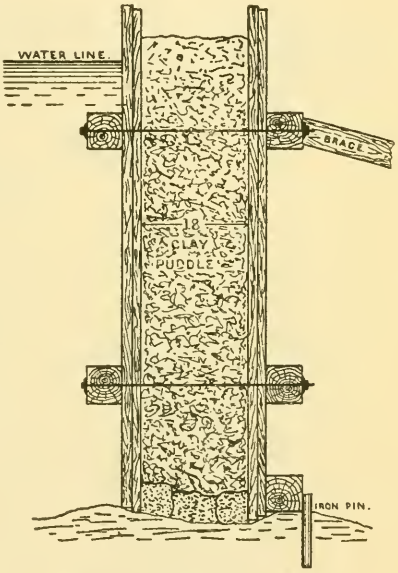


FIG. 4.

head gates in position was built, and water was turned in against the gates by July 18th, although the coffer-dam was not removed until August 14th. Fig. 5 shows arrangements of head gates, forebay, racks and flume. The arrow indicates the direction of the water as it flows in the race. *A A* is the race wall with an opening in front of the forebay 21 ft. wide, which is spanned by three 16-in. steel I beams, supported in the center by an iron column *C*. The opening is widened out on the inner face of the race wall to 26 ft., so as to give easier

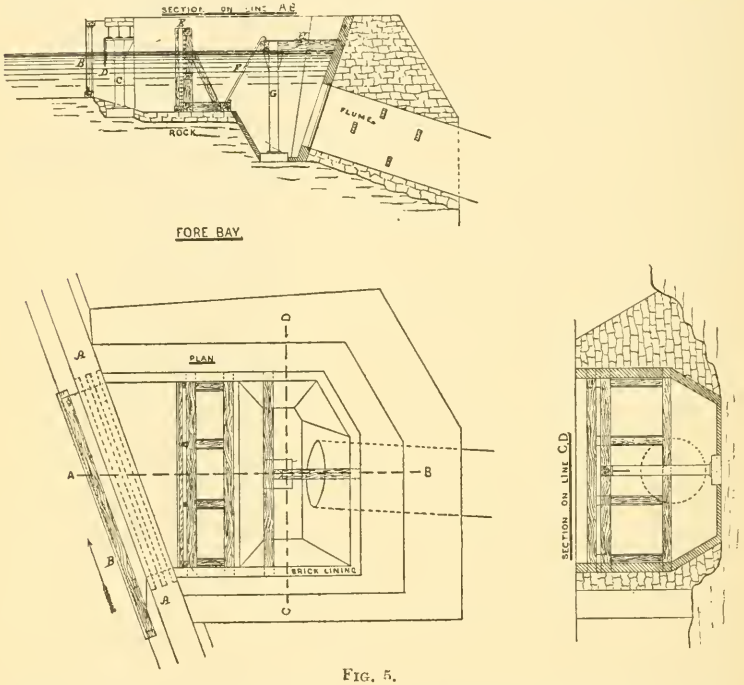


FIG. 5.

entry of water unto the forebay. This opening is guarded by coarse rack *B*, made of 1¼-in. round iron bars spaced 6 ins. apart, and serving to stop any large pieces entering the forebay. On the face of the column *C* is a bracket supporting a wide plank *D*, 2 ins. thick, which has its lower edge 1 foot below the surface of the water. This keeps out much of the floating débris carried along with the water, such as shavings, straw, leaves, etc. The gates *E* are placed inside the wall, so that they are protected by the building to be erected. Temporarily they are now enclosed in a frame shanty. The fine rack *F* is set at an

angle of 60°, so that in clearing off the rack the débris is drawn and deposited on the platform *H*. Cast-iron column *G* carries a 12 × 12-in. oak timber that holds the head of the fine rack *F*. A No. 10 wrought-iron slush pipe *A*, Fig. 2, is carried along the top of the flume outside, terminating below the floor of the wheel pit, and is designed to carry the débris raked from the fine rack directly into the wheel pit. Fig. 6 shows head-gate construction in detail. All the gate frame is made of

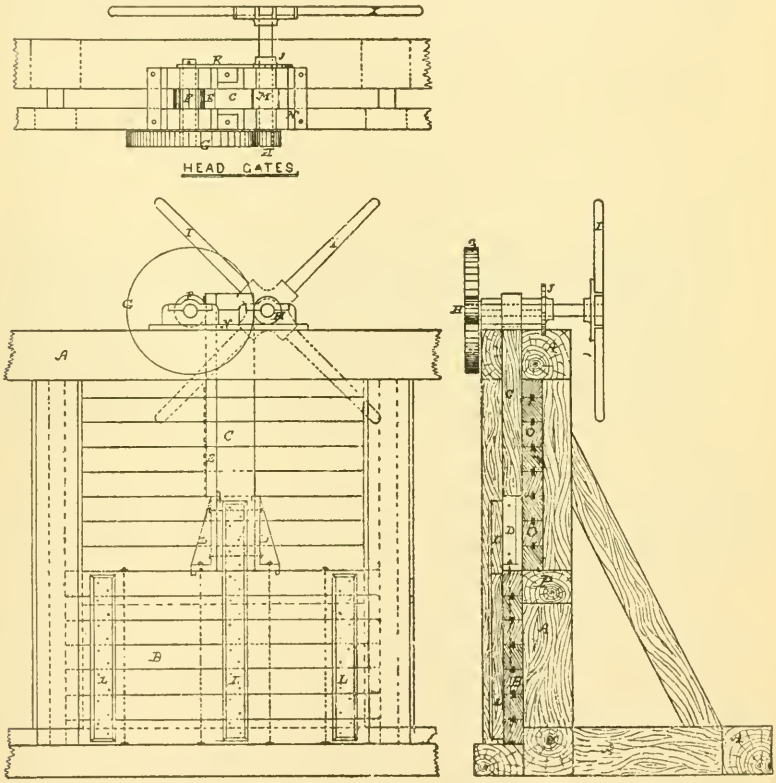


FIG. 6.

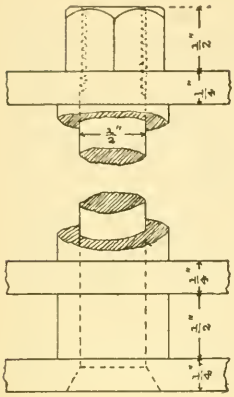
12 × 12-in. white oak, fitted with double mortise and tenons, and all bolted and put together at joints with red lead and oil. The gates *B* are made of seasoned white oak plank, 5 ins. thick and 6 ins. wide, with a ¾ × 1¼-in. tongue strip between the planks. These are all jointed with red lead and drawn together with 1-in. bolts and battens *L* secured by ⅝-in. lag screws. There are three gates, each covering a water-way 5 ft. 4 ins. × 2 ft. 6 ins., all the water being admitted below girt *P*. The

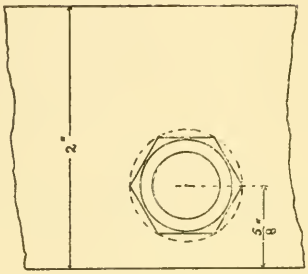
space between the top frame and girt P is filled with the same material and form as the gates themselves. The rack staff C is 5×9 ins. oak and fastened to gate B by the cast-iron brackets D, all thoroughly bolted. On the side of rack staff C is fastened a straight rack E, with the teeth boxed. The bed plate N contains the raising mechanism. The pinion F has 12 teeth, boxed, and is 2 ins. pitch and 4 ins. face, meshing into the teeth of rack E. On the outer end of the shaft carrying pinion F is a spur wheel G, having 55 teeth, $1\frac{3}{4}$ ins. pitch, and $3\frac{1}{2}$ ins. face. Into this gears the pinion H, having 12 teeth. The shaft operating the pinion H has on its other end a spider hub fitted with lever arms I. A loose collar M plays on the shaft and acts as a roller against the back of the rack staff C. A rachet wheel J is keyed on this shaft and engages the pawl K, which retains the gates at any height.

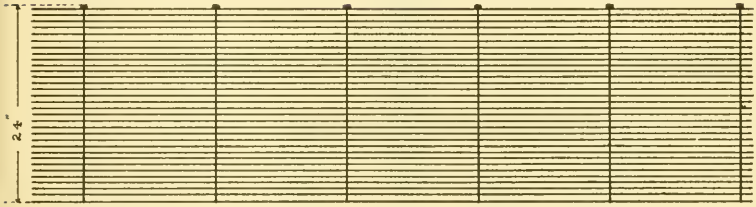
Fig. 7 shows details of fine rack construction. Sections 2 ft. wide are made, so as to allow of easy removal, instead of making the rack all in one piece, as is usually done. The tie rods are $\frac{1}{2}$ in. diameter, with countersunk head let into the first bar of a section. The thimbles are made of $\frac{1}{2}$-in. wrought-iron pipe. Alternately the thimbles and bars are slipped on the bolts, and, when screwed tight together, the $\frac{1}{2}$-in. nut on the end maintains a like spacing. The bars are $\frac{1}{4} \times 2$ ins. in section, and are turned edgewise to a 2-in. inner radius at the upper end, the better to land the rakings on the platform. The holes for rods are drilled $\frac{5}{8}$-in. centers from the back of the bar, so as to allow the rake teeth to clean the rack.

Fig. 8 shows the general arrangement of the engines and water wheels in the power-room. The room is 50 ft. wide by 64 ft. long; the wheel pit, as before stated, is 14 ft. wide by 64 ft. long, leaving the engine floor 36 ft. wide, giving space enough to place four horizontal engines of 500 H. P. each.

The water wheels are twin Poole-Leffel central-discharge turbines, 23 ins. in diameter, and at a speed of 560 revolutions per minute, under a head of 92 ft. 6 ins., develop 500 H. P. each, with a discharge of 3 800 cu. ft. of water per minute. The wheels proper are made of phosphor bronze with buckets of Otis steel, tinned. The wheel bedplates are heavy cast-iron box sections, machined and bolted together with heavy bolts fitting reamed holes. The wheel shaft is $4\frac{7}{8}$ ins. diameter, running in adjustable babbet-lined bearings. A rope wheel







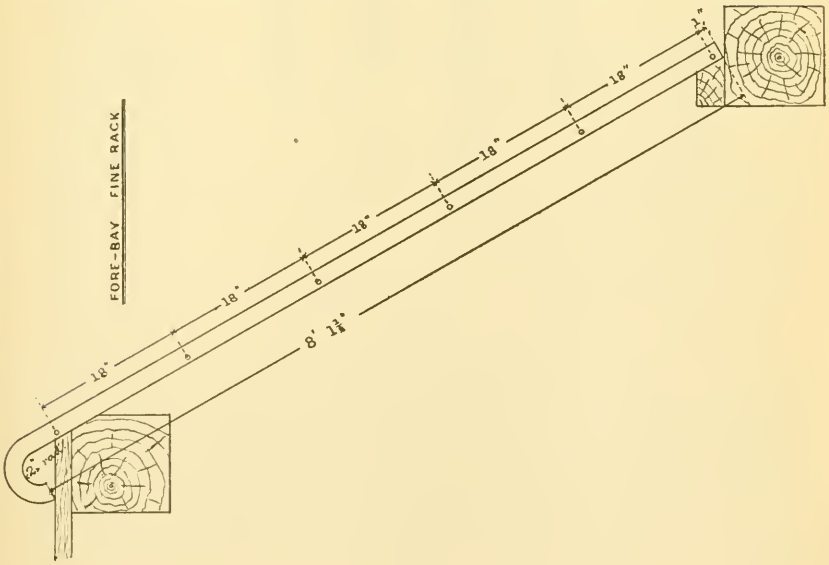


FIG. 7.

4 ft. in diameter is keyed on the shaft, and is grooved for fifteen 1¼-in. Manilla "stevedore" ropes, made with four strands and a core, worked in with plumbago in the process of making. From the 4-ft. wheel, 15 ropes run to a rope wheel on the line shaft above, 76.8 ins. in diameter, and grooved for sixteen 1¼-in. ropes. The rope being endless, the idler strand is passed over a 5-ft. single grooved wheel, placed in a movable frame. The frame traverses in iron guides and maintains by its weight a constant tension on all the ropes. This is made adjustable for the amount of tension, by the application of counter-weights to the frame. The speed of the line shafts is 350 revolutions per minute, and the rope travel is 7 037 ft. per minute.

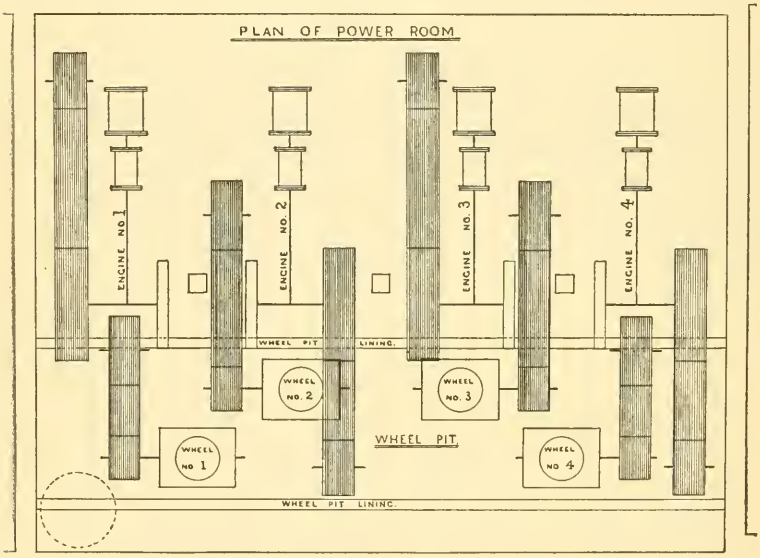


FIG. 8.

The water wheels are supplied from a steel flume 7 ft. in diameter, as shown on Fig. 2. From the horizontal portion of the flume a 4-ft. pipe leads down to each wheel and has a geared 48-in. Chapman valve at the lower end, between pipe and penstock, as shown on Fig. 3. These valves are fitted with a 12-in. by-pass, for the purpose of equalizing the pressure on both sides of the large valve, in opening or closing.

Fig. 9 shows the steel flume. From the forebay to the flange A, at the bottom of the elbow, the plates are $\frac{5}{16}$ in. thick. All below flange *A* are $\frac{3}{8}$ in. thick. The $\frac{5}{16}$-in. portion is all single riveted. The longitudinal seams of the $\frac{3}{8}$-in. portion are double riveted. Bottom head *B* is $\frac{1}{2}$ in. thick and flanged to an internal radius of $1\frac{1}{2}$ ins. Head *C* is $\frac{1}{2}$ in. thick and convexed to a radius of 7 ft. The center of this head is stayed to the second sheet from the end by eight $1\frac{1}{4}$-in. bolts. Bolts are spaced through head on an 18-in. circle. Flanges *A D D* are made of 3 x 3 x $\frac{7}{16}$-in. angles, and are held together by 132 bolts $\frac{3}{4}$ x $2\frac{1}{2}$ ins. Joint packing was made of $\frac{1}{2}$-in. solid round rubber. At the upper end of the flume, where it is built in the forebay, an angle-iron ring of 4 x 4 x $\frac{1}{2}$ ins. is riveted outside and 18 ins. from the end.

This is to form a stop water, as well as an anchor. Besides this, short pieces of 3 x 3 x $\frac{7}{16}$-in. angle are riveted on the outside, to securely anchor the pipe in the masonry. All the variation due to contraction and expansion is absorbed at the elbow. At the junction of the horizontal and vertical portions of the flume two $1\frac{1}{2}$-in. stay bolts E, with nuts and packing on the inside and outside, run through the pipe. These are designed to prevent any dilation or distortion at that point. The flume at bottom head *B* rests upon steel I beams, and upon the rock. The flume complete weighed some 31 tons. All was coated with two coats asphalt paint, inside and out.

A horizontal "Woodbury" compound, condensing, slide-valve engine with extra heavy bed plate, is set in the power-room at point marked in Fig. 8 "Engine No. 1." Steam cylinders are placed with the large cylinder outside, so that pistons and rod may be easily removed. Cylinders are 19 ins., and 31 x 24-in. stroke. At 167 revolutions, with a boiler pressure of 110 lbs. per square inch, vacuum 22 to 24 ins. and cutting off at $\frac{5}{16}$ stroke, the engine is rated at 500 H. P., and is guaranteed to produce a horse-power on an evaporation of 20 lbs. of water per hour. The crank shaft is a steel forging in one piece. Journals are $11\frac{1}{2}$ ins. diameter by 21 ins. long. Crank pin, $11\frac{1}{2}$ ins. diameter by $8\frac{1}{4}$ ins. long. The end carrying the rope driving wheel has an outboard bearing. Governor balance wheel is $8\frac{1}{2}$ ft. diameter by 25 ins. face. Rope driving wheel in halves and 10 ft. 6 ins. diameter and grooved for fifteen $1\frac{3}{4}$-in. ropes. These ropes lead to a 5-ft. rope wheel on the line shaft above, with same arrangement for tightener as is applied to the water wheels.

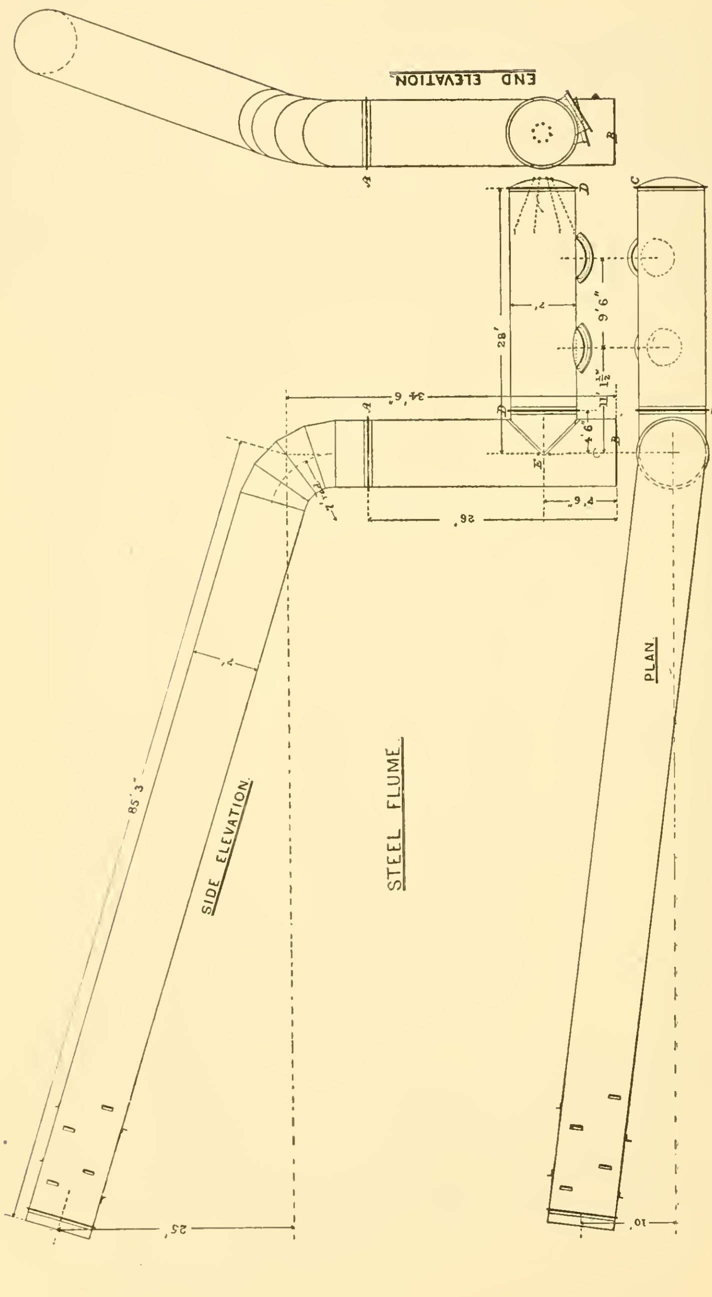

END ELEVATION.
STEEL FLUME.
SIDE ELEVATION.
PLAN.
85′ 3″
34′ 6″
28′
26′
25′
9′ 6″
10′
A
B
C
D
E

Fig. 9.

Fig. 10 is a section of rope driving wheel rim, and gives angle of grooves, pitch and sizes as shown. Rope speed of engine drive is 5 500 ft. per minute.

The condenser applied is H. W. Bulkley's jet condenser and heater combined. It is shown in position on Fig. 3. The injection water is taken from the horizontal portion of the flume, and is supplied under a pressure of 22 lbs. per square inch. The vacuum maintained is 27 to 28 ins. In its performance it is very satisfactory. A strainer box *A*, Fig. 3, is placed in the supply pipe and contains a copper wire strainer of $\frac{3}{16}$-in. mesh, that is arranged to be quickly removed and replaced by a clean one. The strainer is for the purpose of catching any floating matter that might clog the injection jet, and prevent the condensation. An automatic valve *B* is placed above the condenser, so that the engine can operate non-condensing.

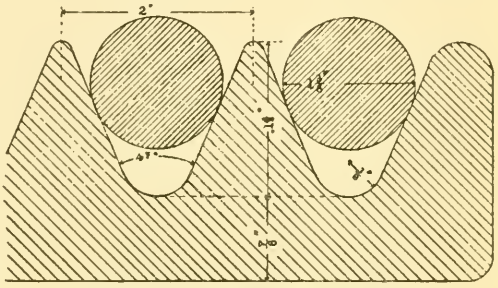


Fig. 10.

The boiler-room and boilers *H* are shown on Fig. 2. The house is designed for ten 200-H. P. boilers, only five of which are now in place, space being left for the other five when needed. They are set in one battery. Boilers are horizontal-tubular, set in brick work. Each boiler is 66 ins. in diameter and contains seventy-two 3½-in. tubes, 18 ft. long. The shell is made of three plates, each long enough to reach entirely around without piecing. All the longitudinal seams are placed so as to be above the reach of the fire. Shell plates are $\frac{13}{32}$ in. thick, and all edges planed for calking. All rivet holes are drilled, and any variation of holes was corrected by reaming, no drift pins being used for that purpose. Heads are ½ in. thick and flanged with an internal radius of 1½ ins. Tube holes in heads were drilled and chamfered on both sides. Tubes are American lap-welded, of charcoal iron, set with expander and beaded over on the outside. The horizontal seams are butt seams, with a plate on the outside 9¾ ins. wide and one on the inside 15 ins. wide, with triple rows of rivets each side of the joint;

all proportioned to the standard of the Hartford Steam Boiler Inspection and Insurance Company. Each head is stayed to shell with solid crowfoot stays of 1-in. square iron, equal in quality to Burden's best iron. Each boiler has a steam drum 33 ins. in diameter by 38 ins. high, and is supported in brick work by four lugs near the front end and four near the rear end. The rear lugs rest on iron rollers 1 in. in diameter on suitable plates set in the brick work. Front lugs are built solid in the brick. Calking is the Connery system. The boilers are intended for a pressure of 110 lbs., and were proved with a hot-water pressure of 180 lbs. per square inch. The feed water enters the boilers through the front head above the tubes, and is distributed in the boiler through a perforated $2\frac{1}{2}$-in. pipe. Each boiler is equipped with water column and gauges, pop safety valve, asbestos-lined blow-off cock, and all the necessary water-stop and check valves, so that the boilers may be used independently, or as one battery. Each boiler has two manholes, one on top and one in the front head below the tubes. All the material in the plates and heads is Park Bros. open-hearth, homogeneous flange steel, with a tensile strength guaranteed between the limits of 55 000 to 62 000 lbs. per square inch of section, and limit of elasticity between 35 000 and 40 000 lbs. per square inch. The boilers are supplied with water by a duplex Worthington pump, with 9-in. steam cylinder, $5\frac{1}{4}$-in. water cylinder by 10-in. stroke, taking the water from the flume through a strainer-box. The boiler house is paved with hard brick, laid herring-bone on edge, grouted with cement mortar, and bedded on 8 ins. of concrete. The smoke jack is of No. 10 iron, built on a skeleton frame of $2\frac{1}{2} \times 3 \times \frac{5}{16}$-in. angles. A damper is arranged from each boiler in smoke jack. As shown, it is graduated in size, being 5 x 10 ft. at the chimney end. It was painted with two coats of asphalt paint inside and out.

Fig. 11 shows plan of line-shaft floor. Shafts are of hammered iron 5 ins. in diameter, and arranged with heavy floor pedestals, fitted with self-adjusting, ring-oiled, babbet-lined bearings. The rope wheels are placed on heavy cast-iron quills, furnished with Hill friction clutches of 500 H. P. capacity each. By a series of jaw clutches, pulleys and belts any line shaft can be operated from any water wheel or engine, all the line shafts making the same number of revolutions.

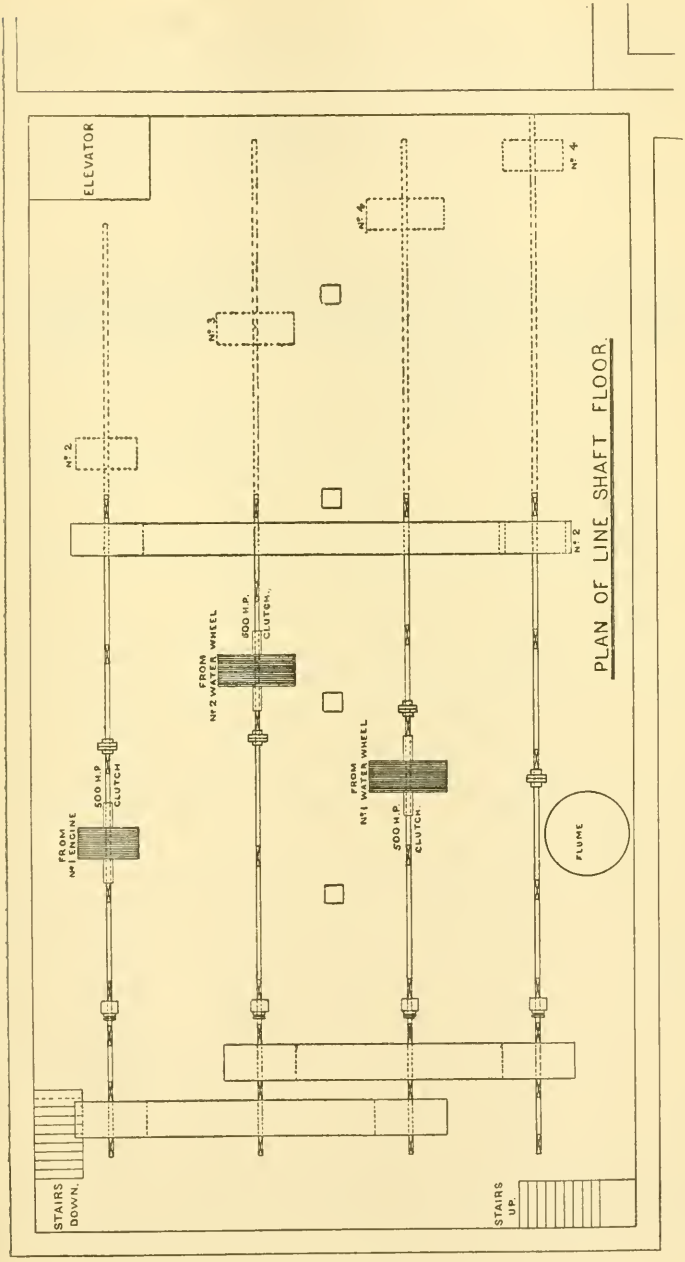

ELEVATOR
Nº 2
Nº 3
Nº 4
Nº 4
Nº 2
PLAN OF LINE SHAFT FLOOR.
FROM Nº 2 WATER WHEEL
500 H.P. CLUTCH.
FROM Nº 1 WATER WHEEL
500 H.P. CLUTCH.
FROM Nº 1 ENGINE
500 H.P. CLUTCH
FLUME
STAIRS DOWN.
STAIRS UP.

FIG. 11.

Fig. 12 shows in detail the quill, clutch and bearings. The floors of the line-shaft and dynamo rooms are made of steel I beams, built and anchored into the walls at outside ends, and attached in the center to

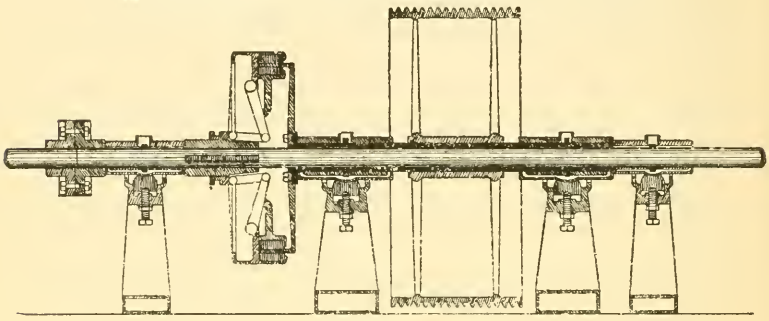


Fig. 12.

longitudinal girders of heavy steel I beams. Girders are supported on columns formed of channel beams and lattice bracing. Columns rest on cap-stoned masonry carried down to the rock.

The wooden floors are made of 2 x 6-in. hemlock plank, matched and surfaced and spiked to fitting strips under flange of I beams. Upon this, and laid at right angles to the hemlock, is nailed $1\frac{1}{8}$ x 4-in. matched, seasoned hard maple.

All of the stone used in the work was quarried stone, as the rock cut out was a shale that disintegrated on exposure, and was fit for nothing but filling.

The brick were hard-burned sewer brick, wetted as they were laid.

The cement used was the Columbia brand of American Portland. The result of five tests of this cement, made by O. H. Peacock, Esq., Engineer of the East Side Sewer Commission, of Rochester, N. Y., is appended.

	Briquette made.	Put in water.	Broken.	Tensile strength per square inch.	Time of immersion.
				Pounds.	
1...	Aug. 24, 1892, 3 P. M.	3.30 P. M.	Aug. 25, 1892, 3.30 P. M.	225	24 hours.
2...	Aug. 24 1892, 3 P. M.	3.30 P. M.	Aug. 25, 1892, 3.30 P. M.	255	"
3...	Sept. 12, 1892, 10.25 A. M.	10.55 A. M.	Sept. 13, 1892, 11.30 A M.	210	"
4...	July 19, 1893, 10.10 A. M.	11 A. M.	July 26, 1893, 11 A. M.	446	7 days.
5...	Aug. 21 1893, 3 P. M.	3.30 P. M	Aug. 28, 1893, 3 P. M.	535	"

All of the rock cutting, excavating and stone and brick work was performed by the firm of Chambers & Casey, of Rochester, N. Y., they also furnishing all the materials and tools to complete the buildings.

The water wheels were built and erected by the firm of Robt. Poole & Son Company, of Baltimore, Md.

The engine and boilers were from the Stearns Mfg. Co., of Erie, Pa.

The steel floors, roofs and flume were furnished by the Rochester Bridge Works, of John F. Alden.

Line-shafting bearings, friction clutches and pulleys were supplied by the Hill Clutch Company, of Cleveland, O.

The carpenter work was performed by R. Williamson; the windows, doors and frames were from the Ocorr & Rugg Company; the smoke jack and slush pipe and roof covering were furnished by the John Siddons Company; the steam piping, pumps and plumbing by Barr & Creelman, all of Rochester, N. Y.

DISCUSSION.

William P. Craighill, Prest. Am. Soc. C. E.—I did not expect to say a word concerning the interesting paper presented by our friend Mr. Cartwright, but, before the discussion begins, I will take the liberty of emphasizing several points he has made.

One is the great care which should be taken in the construction of brick masonry in order to insure good results. Two cases may be mentioned which were a part of my experience more than 40 years ago. They are of a special and unusual character. I refer to Fort Pulaski, on the Savannah River, and to Fort Sumter, in Charleston Harbor.

These forts were then unfinished. Later, in the Civil War, much of their masonry was greatly broken to pieces by the guns of General Gillmore. After the war, when repairs and reconstruction began, the importance was very clearly shown of the truth of one of Mr. Cartwright's cautions, viz., the necessity, in order to have the best brick masonry, of great care in seeing every brick carefully soaked in water before being laid. It is not enough to have the bricks simply damp. They should be soaked so as to be dripping when laid. And, to this end, every mason should have his barrel or tub of water near him in which his bricks may be soaked before being used, and an inspector should be around to see it properly attended to. The result proved again the importance of this precaution, for in the masses of masonry

at Forts Pulaski and Sumter, broken up under those peculiar circumstances, it was found that the adhesion of the mortar to the brick was greater than the cohesion of the particles of the brick among themselves.

I recollect well how this watering of the bricks was insisted on by my then chief, General Totten, in his detailed instructions to his young subordinates. The farther south I went, as, for instance, at the Dry Tortugas, in the Gulf of Mexico, the more stress was laid upon it.

There is another point to which Mr. Cartwright referred, the use of salt in masonry in freezing weather. This may be of great value in special cases. Take, for instance, one like the Great Kanawha River, where, in building locks and dams, much of the work must be done in coffer-dams. The Kanawha is a very uncertain, inconstant stream. The freshets come often and unexpectedly, and sometimes very high. To keep them out would require coffers of very great height with proportionate expense and inconvenience of use at lower stages. Advantage should therefore be taken of every hour when the coffers are not flooded, to expedite the work. The masonry on the Kanawha is of stone and concrete.

There is one more point to which I will refer, the danger of too much dependence on mathematical formulas. Our education as engineers leads us, and very properly too, to value our mathematics and formulas most highly, and the man who undervalues them, or tries to dispense with them, is a very unwise one. But we should avoid both extremes, and this is a safe rule in almost all departments of life.

In hydraulic work of the special kind to which most of my attention has been given for many years past, the improvement of rivers and harbors, there are so many conditions to be taken into account, which are very variable, even from hour to hour, and so many of which we know little, and about which we must accept mere theories, that the formulas must be used with great care in order to avoid serious mistakes. Even when we are applying formulas to cases like masonry, bridges, etc., where the materials are few and well known and we have varied experiences, we find it necessary to have large factors of safety.

I will not detain the Society longer from the discussion of Mr. Cartwright's interesting paper, and we will now be glad to hear any one who may wish to speak on the subject.

H. W. Brinckerhoff, M. Am. Soc. C. E.—I would like to ask Mr. Cartwright how he got along with his masons with the use of very wet bricks? The masons, I know, complain that very wet bricks soften their fingers.

Bernard R. Green, M. Am. Soc. C. E.—Having, unfortunately, but just come in, I did not hear the paper read, and perhaps am out of order; but I do not understand why the bricklayers' hands should have been so much injured in laying wetted bricks in cement mortar.

I have laid many millions of bricks in that way and never observed serious results of that kind. The bricklayer does not necessarily put his hands into the mortar. He handles the clean, wet brick with one hand and the mortar with the trowel in the other. The bricks should be well dampened, but not soaked. The experience referred to, however, by the author may have been in some other sort of work than I have supposed.

Charles E. Emery, M. Am. Soc. C. E.—I had intended to say but a few words. The paper, to a certain extent, explains itself so entirely that there is little opportunity for discussion. I have seen one of Mr. Cartwright's plants in operation at Rochester in which rope transmission is used, and it operated very perfectly, and was a very satisfactory piece of work. The very thorough way in which Mr. Cartwright does his work is to be commended, and I am sure we must feel very grateful to him for the careful manner in which he has presented an example here.

Referring to the criticism of formulas brought out in the discussion, I will say that the fault of some formulas is that they are not correctly founded on facts, or are incorrect in form. A correct formula, with correct constants, based on careful experiments, will always give correct results. The difficulty is that the mathematicians and physicists do not always take proper pains to obtain correct constants, while practical men who attempt to formulate experimental results frequently attempt to construct the formula too simply, so that it is little more than the rule of three, and thereby make it inapplicable, except for conditions closely approximating the experimental ones. I have never had any difficulty in solving questions in regard to expansion by simple calculations. Such calculations frequently require the combination of rules relating to linear expansion with those for the transverse strength of girders, but the method is evident; there should be no difficulty in ascertaining just how much pipes can be permitted to bend before becoming over-strained. It was only a few days ago that an engine-builder asked my advice about connecting the horizontal cylinders of a cross compound engine rigidly, by means of piping under the floor. They had been accustomed to make such connections in the smaller engines, but in this case the superintending engineer objected. Simple calculations showed that if it were possible to hold the two engine foundations rigidly in position, the compression would not be beyond the limit of elasticity of the metal of the pipes, even if the whole expansion were taken up in that way; but there would be great strain on the joints, which might become leaky when the pipes were cooled. Even if metallic joints were used the compressive strain caused by the temperature would amount to very many thousand pounds per square inch, based on well-known facts as to the rate of expansion and the modulus of elasticity, and I called the attention of

my client to the fact that the separate foundations were not laterally sufficiently rigid to resist those strains, but that the weaker side would be pushed over slightly, thus throwing the engine out of line. As this was a sensitive point, and as it was desired that the engines should, above everything, work smoothly and continuously, it settled the question, and on looking over the connections it was found that the pipes could be very easily crooked and thus provide for expansion without an expansion joint.

Mr. Cartwright studies his work so carefully and adapts means to ends in so thorough a way that I regret no figures as to costs are given. This is an important omission. Some of us are making special studies on the general subject from an economic basis, in which are considered, not simply the coal consumed, the amount of labor employed and the cost of repairs, but the first cost of the plant as a whole, for the reason that the interest thereon, which is an annual expense, may under certain conditions be so great that an apparatus cheaper than another may, when all expenses are considered on an economic basis, be the best adapted for the work. I will be pleased to hear from Mr. Cartwright the cost of his building, wheel pits, head and tail races, and, in connection with them, the cost of installing the hydraulic plant, the cost of installing the steam plant, the cost of shafting, pulleys, ropes, etc., used in transmission, and it would be more complete to have the cost of the dynamos added, when, by a comparison of these facts with those known in other cases, the engineer is best able to judge of the value of the installation from a commercial standpoint.

Robert Cartwright, M. Am. Soc. C. E.—My experience in hydraulic work has been that wet brick softens the skin so the constant attrition with the brick very soon removes it, and sore fingers is the result. I always provide my men with rubber finger covers, and keep a supply on hand for use at all times. If I cannot get the rubber covers, I buy leather harvesting mittens for the purpose.

I differ entirely with Mr. Green as to the brick being only "damp." I want them "soaked." When work is laid with soaked brick with hydraulic mortar, as our President has truly said, "the adhesion of the mortar to the brick is greater than the cohesion of particles of the brick among themselves." I do not expect, in building hydraulic masonry, to run the wall up as rapidly as with dry or damp brick, as the work would be apt to "bulge," but judgment in carrying the work on enables the engineer to shift his men on the work, giving the mortar time to harden before overloading it.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

699.

(Vol. XXXI.—March, 1894.)

THE RENEWAL OF THE CHANNEL PIER OF THE CINCINNATI AND MUSKINGUM VALLEY RAILWAY BRIDGE OVER THE SCIOTO RIVER.

By MORTON L. BYERS, Jun. Am. Soc. C. E.

READ MARCH 21ST, 1894.

WITH DISCUSSION.

Among the structures damaged by the floods of the spring of 1893 was the east pier of the single-track bridge of the Cincinnati and Muskingum Valley Railway, across the Scioto River, south of Columbus, O.

In July, the writer, on taking charge as Engineer of Maintenance of Way of the Cincinnati and Muskingum Valley Railway, found that false work had been driven to relieve the pier of the weight of the bridge, plans prepared and bids requested, and he was informed that the probable cost would be about $3 500 for rebuilding.

The plan, on examination, proved to be for the ordinary design of masonry pier, resting on a foundation composed of three lines of piles driven about 4 ft. between centers, longitudinally, capped trans-

versely, and with a grillage of two solid courses of 12 x 12-in. hard-wood timber drift-bolted to the caps. From base of rail to low water-mark was 28 ft. 9 ins., and to the top of the grillage foundation was 10 ft. 3 ins. additional, making 13 ft. 3 ins. from extreme low water-mark to the top of the piling, which was to be 18 ft. long in place.

At various times the river had threatened to undermine this pier, and, as a protection, a ring of piling had been driven at a distance of about 25 ft. from the pier, entirely around it, and the space between the piling and the pier had been filled with rip-rap and blast-furnace slag.

The last flood had succeeded in dislodging this material from the upper end of the pier, and had undermined the nose to some extent. When the water subsided, the pier suddenly cracked from top to bottom, and commenced to settle badly, so that it became necessary to drive temporary bents at once, to carry the two spans (one a Howe and the other a Pratt truss), of 148 ft. each, until such time as the pier could be rebuilt.

Early in July a single bid was received, and, when opened, proved to be for $8 500, much to the writer's surprise, as he had not up to this time examined the structure. On going to the bridge, to investigate into the cause of this unexpectedly high bid, the following state of affairs was found : The temporary bents carrying the two spans were arranged as shown by Plate XLIV, the bents on either side of the old pier being but from 12 to 13 ft. apart, and not quite parallel. The space between the pier and the old piling had been refilled with rip-rap and boulders of all sizes.

At a depth of about 18 ft. below low water a bed of hard blue clay appeared, while in the vicinity of the pier, and carrying it, was a mound of sand and gravel, which extended to the surface of the water and was about 80 ft. in diameter at its base. Beyond the base of this mound the water had cut, in places, entirely through the gravel and exposed the clay.

A boat had been built for use in driving the piling for the temporary bents, and proved to be 40 ft. long by 14 ft. wide—just too wide to be used in working between the bents when the pier should be removed.

It was evident that if the plan already made was to be carried out, a coffer-dam of some sort was a necessity. But what kind, and

where? It was apparent that it must enclose all the temporary bents, which would require a dam about 65 ft. square, or about 250 ft. in length, with piles at least 28 ft. in length. With the dam in place, there was a strong probability of heavy expense for pumping. Also, the removal of material by derrick and boxes would be very slow and expensive, being much interfered with by the lack of room between bents. Under favorable circumstances, it would take seven or eight weeks to obtain material, drive and pack this dam, and the excavation would require about three weeks more before the piling could be driven; so that it would be well on to the last of October before we could be sure of being out of danger from high water. This was a very important feature, as the pier upheld the channel span; the water when high struck it obliquely, forming a dangerous eddy for a coffer-dam to be exposed to, and low water after the middle of September could not safely be counted on.

An estimate of the probable cost of the work was made as given below:

Estimate of Cost of Replacing Pier—Plan A.

Removing old pier	$200 00
250 lin. ft. pile coffer-dam, at $10	2 500 00
400 cu. yds. excavation, at 70 cents	280 00
Pumping, including handling of machinery	1 500 00
27 piles in foundation, at $10	270 00
10 500 ft. B. M. grillage, at $25, in place	262 50
329 cu. yds. masonry, at $7.50 per cu. yd	2 467 50
Total	$7 480 00
Add 10% for contingencies,	748 00
Total estimated cost	$8 228 00

While it was, of course, possible, and even probable, that the work could done under this estimate, the fact remained that no contract could be made even at that figure.

To effect an economical solution of the problem, it was evidently necessary to devise some plan for dispensing with the coffer-dam entirely. With the excavation once made, the piling could be driven

and sawed off to a level with a circular saw on a vertical shaft, and a crib sunk on the piling, or very probably the bottom would be found sufficiently secure to dispense with the piling entirely, using an ordinary crib foundation. But the excavation was just what presented the difficulty.

As soon as the old pier was removed, the rip-rap and other loose material would naturally slip into the hole so made, and there was not room to remove this material till the pier was removed. So any machine used for excavating must be capable of handling such material, and also any boulders that might be encountered in the gravel itself.

A dredge small enough to work between the bents could not be obtained, and, even if it could, the work was not of sufficient magnitude to admit of the expense of transportation, or of the purchase of any expensive special tools.

Finally it was resolved to use a scraper made after the plan given in Fig. 1, and of the dimensions there given. It was made at the

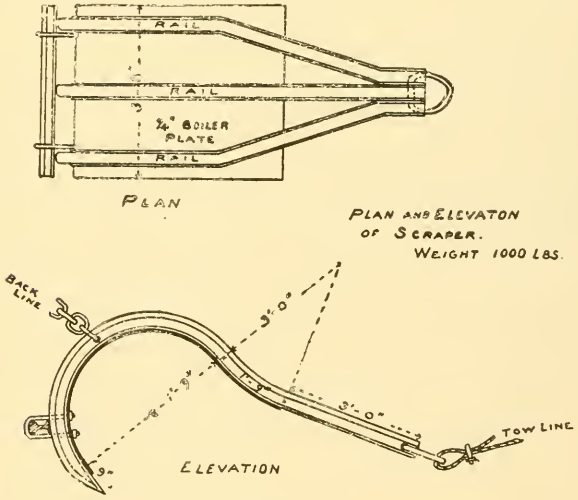
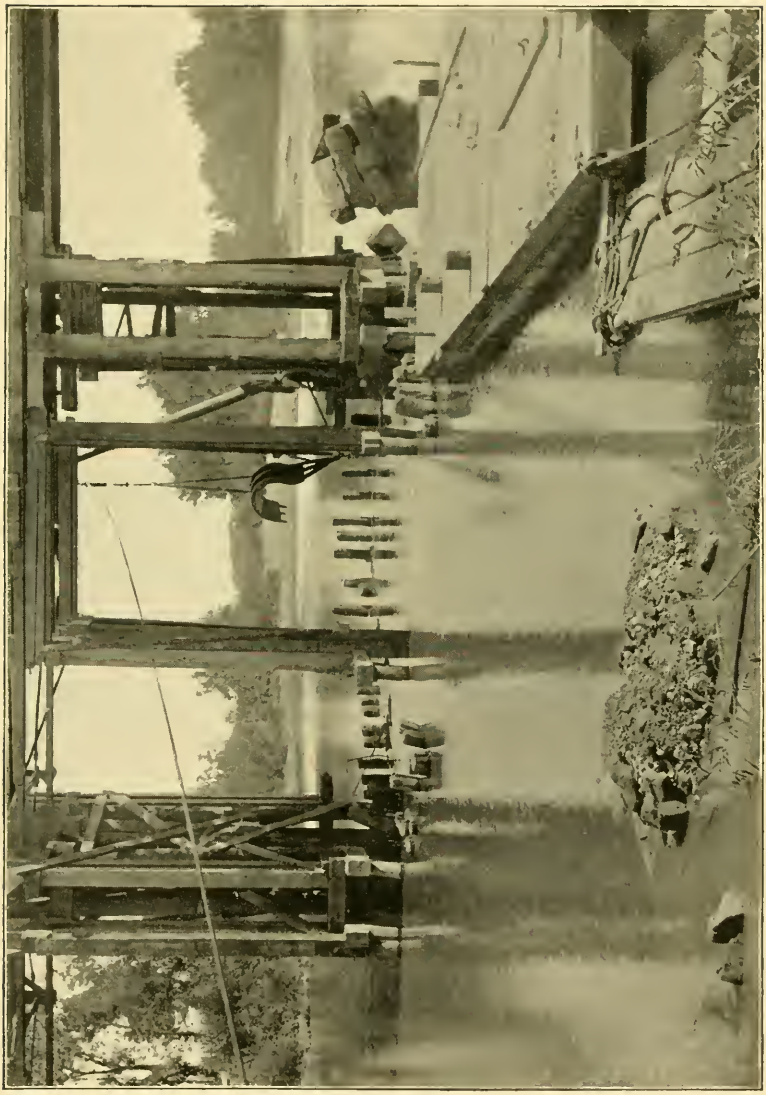


FIG. 1.

railway shops of old boiler iron, strengthened by three ribs of light iron rail, and was operated by a Mundy 20-H. P. two-drum hoisting engine, mounted on an island that had formed just above the pier. The towing-line was run direct from one drum to the back of the scraper, and the back line from the other drum over a sheave on the bridge to



the front of the scraper (see Plate XLIV), thus giving the engineer entire control of the work.

In order to allow the scraper to work, it was necessary to remove 13 of the old piles forming the upper end of the ring. The piles on either side of the one to be removed were sawed off level, and a 20-ton hydraulic jack was placed on each. Across the tops of the jacks was placed a 12 x 12-in. timber, and over it a heavy switch chain was placed, and secured to the pile to be pulled. The links of the switch chain were $\frac{5}{8}$ in., and they were tested to the utmost. Most of the piles were 18 ft. long, bedded about 10 ft. in the gravel. One was 20 ft. long, bedded 12 ft., and it broke the chain twice.

The dredging was begun on August 19th, and on September 1st a clear depth of 12 ft. had been gained over the entire foundation, somewhat over 550 cu. yds. of material having been removed. The scraper at first showed a decided tendency to upset whenever it came in contact with a piece of rip-rap, but this was overcome by bolting a piece of rail across the end, a few inches above the teeth, as shown in Fig. 1. After this was done it rarely upset, and nearly always brought out all the gravel it would hold.

The boiler capacity of the engine used proved much too small, the time required to get up steam and replenish the water being fully four times that employed in actual excavation. With proper appliances in this respect, and with the weight on the toe of the scraper, as before explained, this machine would have completed the excavation in not over three days. But even as it was, it kept from two to four men busy shoveling to one side the material dragged out. Plate XLIV shows the scraper suspended by the back line ready to let it drop for a haul-out. The material handled was not generally large, although stones weighing from 300 to 400 lbs. were sometimes brought out. The towing line used was a $\frac{3}{4}$-in. hemp-center wire rope, and the back line a $\frac{1}{2}$-in. line of the same character. Experience proved that 1-in. and $\frac{3}{4}$-in. rope, respectively, would have been better, as both of those used were completely worn out, and could not have been made to do another day's work than they did.

After reaching a depth of 12 ft. throughout, it was decided to dispense with the piling, and a crib composed of six courses of 12 x 12-in. timber, arranged four timbers longitudinally and seven transversely, so as to form three lines of pockets, each 4 x 2 ft. 6 ins., open top and bot-

tom, was built in place directly over its final position. The middle pockets were provided with slat bottoms, and filled with cobble-stones from the river bank. Two-inch tongued and grooved siding was nailed to the top course of the crib, and braced, for a coffer-dam, and the crib was sunk by placing rails on the top of the coffer thus made.

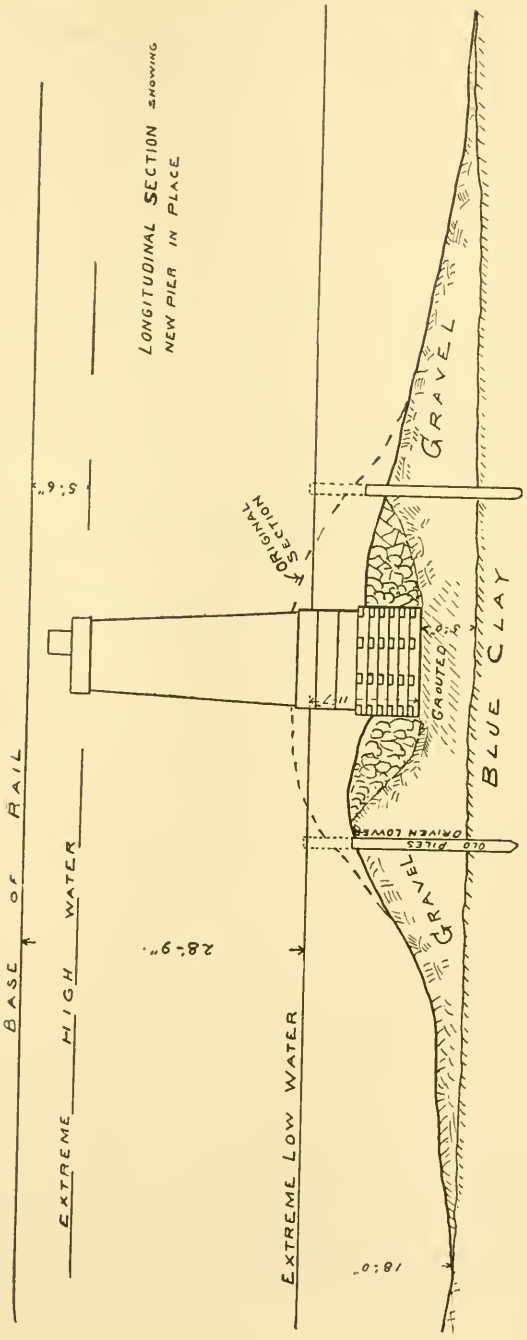
The scraper had left the floor of the excavation quite even, but to cut down all irregularities a rail 11 ft. long was dragged back and forth over the bottom. Of course, the bottom, in some places where the scraper had cut too deep, was much softer than where the gravel had not been disturbed, and it was for this reason that no bottoms were put in the outer pockets of the crib.

After the crib had been sunk it was found to be but 1 in. out of level. A slight ridge of gravel and sand was then thrown up entirely around the crib, and of such depth as to about cover the first course of timber.

In each of the outside pockets was then placed about ½ cu. yd. of rich mortar, followed by an equal amount of concrete, which latter was thoroughly rammed with an iron rammer, weighing not far from 80 lbs.. with a base about 4½ ins. square, the object being to force the mortar under the edges of the crib, and also into the gravel wherever it had been loosened. The center pockets were then treated with a rich grout (it will be remembered that they were filled with cobble-stones), and the outside pockets also filled with cobble-stones and a much weaker grout. In all, 93 bbls. cement were used in the crib alone. It contained 18 pockets, each 4 x 2 ft. 4 ins. x 6 ft., 1 008 cu. ft. in all. Judging from the specific gravity of a pile of cobble-stones, one-third of this space was voids; or it required 336 cu. ft. of grout to make the mass solid. One hundred and forty barrels of sand at 3¾ cu. ft. per barrel were used, making 525 cu. ft., disregarding the volume of cement used. This would show that at least 189 cu. ft. of voids in the underlying gravel were filled by this grout.

In proof, no settlement could be detected after the third course of stone was laid, by which time the cement in the foundation had set.

In building the crib, one mistake was made, that cost about $100, and a delay of about five days. It was thought that the cement used would so close all openings in the bottom and sides of the crib as to make it water-tight, and no attempt to calk the sides of the crib was made. The writer is still of the opinion that this would probably have


LONGITUDINAL SECTION SHOWING NEW PIER IN PLACE
ORIGINAL SECTION
GRAVEL
BLUE CLAY
GRAVEL
GROUTED
OLD PILES DRIVEN LOWER
BASE OF RAIL
EXTREME HIGH WATER
EXTREME LOW WATER
28'-9"
18'-0"
5'-6"

been all that was necessary if it had been allowed to set, but unfortunately the foreman in charge became impatient, and, starting the pumps 12 hours after the grouting had been completed, soon had the crib leaking badly.

In order to avoid increasing the pumping plant, after a second attempt to fill the crevices with cement, sand and dirt were piled around the crib till the water was cut off. The water was then easily pumped out and the masonry laid. It was commenced September 19th, and completed October 11th, no especial effort being made to hasten the work after it was above ordinary low water. It may be of interest to state that the pumping plant consisted of three ordinary barge pumps.

When it was decided not to adhere to the old plans, a bid was received at the following prices, work not to be commenced for 60 days :

"For excavation to a depth of 10 ft. below low water, $3 000" (as it was carried to a depth of 12 ft. this would have been $3 600).

"For furnishing and placing all timber for crib, grillage, etc., $25 per 1 000."

"For removing and rebuilding pier, $10 per cubic yard, contractor to furnish stone."

Items of Cost.		Actual Cost.	Bid.
Removing Old Pier		$189 13	
Excavation :			
Placing engine	$51 65		
Building and placing scraper	54 30		
150 ft. of ¾-in. wire cable	26 40		
Labor excavating	208 71		
		341 06	$3 600 00
Foundation :			
Timber	$252 34		
Framing and sinking crib	113 95		
Concreting and pumping	309 40		
93 lbs. cement	74 40		
		750 09	450 00
Masonry :			
260.71 cu. yds		1 955 32	2 607 10
Total		$3 235 60	$6 655 10

It was decided, however, to proceed with the work at once, and the contract for the masonry was given to Mr. F. C. Neeb, of Lancaster, O., to whom, it is but fair to state, the writer is indebted for many valuable suggestions, the final plans being the result of a careful study

and discussion of the situation. Second-class masonry was decided on at $7.50 per yard, and the foundation was put in on force account, the railway furnishing the carpenter force.

The actual cost of the work is given in table on page 368, and to show the economy of the method employed a comparison is given, using the same quantities and the prices of the bid referred to on page 368.

This does not represent the cost of work performed under exceptionally favorable circumstances. On the contrary, the method employed was entirely new to all parties interested, the machinery, as before explained, was insufficient and unequal to the work, and the necessity for experiment before arriving at the best means delayed the work considerably.

The writer is satisfied that under similar circumstances the cost could be reduced.

If it had been so desired, the excavation could have been carried at least 6 ft. deeper with ease, and at about the same cost per cubic yard, viz., $\frac{341.06}{550} = 62$ cents.

Indeed, as $105 95 of the total of $341 06 is the cost of placing the engine and building the scraper, it is probable any further excavation would have decreased the cost per cubic yard.

In case no island had been at hand, the engine could have been mounted on a boat and anchored in a convenient spot below the pier, gaining the aid of the current, which caused some little annoyance on the work described, by carrying the finer matter back into the excavation.

DISCUSSION.

S. Munch Kielland, M. Am. Soc. C. E. (by letter).—I have read the advance copy of Mr. M. L. Byers' paper, on the removal of the pier of the Cincinnati and Muskingum Valley Railway Bridge over the Scioto River.

Mr. Byers' scraper appears to be a good construction, and ought to be a very practical tool for similar excavations.

I am not able, from the reasons given, to see why the pile foundation as suggested by the former engineer was not adopted. If the scow was too wide, it could not have cost much to have made it narrower ; but, besides, the so-called island appears from the drawing to

be high enough and large enough, together with the old piles, to have been used as support for the pile-driver. Further, why was it necessary in this case to cut off piles at 10 to 12 ft. below low water? Four feet would have been plenty, which would have brought the platform 1 to 2 ft. below extreme low water, and certainly a pile foundation like this would in this case be safer than the crib which was put in. The pile foundation would require very little excavation and less masonry, the repairs could have been made quicker and cheaper, and no coffer-dam would have been needed.

Morton L. Byers, Jun. Am. Soc. C. E. (by letter).—The reasons for using a crib instead of a pile foundation were briefly as follows :

Owing to the positions of the chords, floor beams, stringers and false work, the driving could not be done from the bridge. There was but 21 ft. 9 ins. clear from low water to false work carrying the pedestals of bridge. Of this space would be required, for rigging of driver, 2 ft. 6 ins.; for hammer, 3 ft.; at least 1 ft. fall to start driving, and since a 3-ft. rise could be expected at any time, that much would also have to be allowed for, making in all 9 ft. 6 ins. not available of this 21 ft. 9 ins. Assuming that the piles were to be driven 4 ft. below low water, and excavation carried but to this depth, this would give a pile 21 ft. 9 ins. 21 ft. 9 ins. — 9 ft. 6 ins. + 4 ft. = 16 ft. 3 ins. in length, which, of course, would have been entirely unsafe, as a glance at the cross-section will show. I do not consider a pile under 30 ft. in length as capable of giving a safe foundation with top at the depth mentioned. If a 30-ft. pile was used, it would be necessary to excavate to a depth of 17 ft. 9 ins. in order to get the pile in the leads of the driver.

Again, supposing the excavation made (it is doubtful if it would have been safe to excavate so deeply on account of the false work), it would have required ten days at least before the last pile could be driven, and meanwhile there would be constant danger of the excavation filling in. In this case the pile-driver would have seriously embarrassed us in using the scraper to re-excavate.

As it was, the crib was sunk and secured 36 hours after the excavation was completed.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

700.

(Vol. XXXI.—March, 1894.)

DRIVEN WELLS OF THE PLAINFIELD WATER SUPPLY SYSTEM.

By L. L. TRIBUS, Assoc. M. Am. Soc. C. E.

READ FEBRUARY 21ST, 1894.

WITH DISCUSSION.

General questions, like the sanitary requirements, fire protection needs, and usual methods adopted for the supply of potable water under pressure, have been too exhaustively treated to warrant any time being given to them here. It is, however, often a matter of more or less interest to know some of the special features in particular localities; therefore, the writer, who was resident engineer during construction, ventures to offer to this Society a few notes on the construction and operation of the Plainfield (N. J.) Water Works.

These works may be classed among the "driven well" systems, of which we have comparatively little published information.

Mr. Charles B. Brush, M. Am. Soc. C. E., was Chief Engineer during construction. Mr. Amos Andrews is at present Superintendent of the company. To their courtesies the writer is indebted in the preparation of these notes.

By referring to the topographical map of that portion of New Jersey (Plate XLV), there will be noted just east of the Plainfield water-shed (indicated thereon by broken lines) the surface evidences in the distorted contours of the termination of a glacial moraine.

The porous nature of the under strata in some portions west of this eastern boundary, with their freedom from sand or other fine materials, suggests the gravel and small boulders of the under side of such a moraine, deposited as the larger masses on top passed onward, while the varying character and positions of the strata, sand, clay and gravels at different points throughout the valley, determined by borings for private wells, verifies the surmise.

The region itself is a comparatively level valley, some 7 miles long and from ¼ to 2 miles wide, is fairly well wooded and slopes gently to the westward. It is divided by a small stream running to the southwest, having several short tributaries; together they furnish excellent surface drainage for the city.

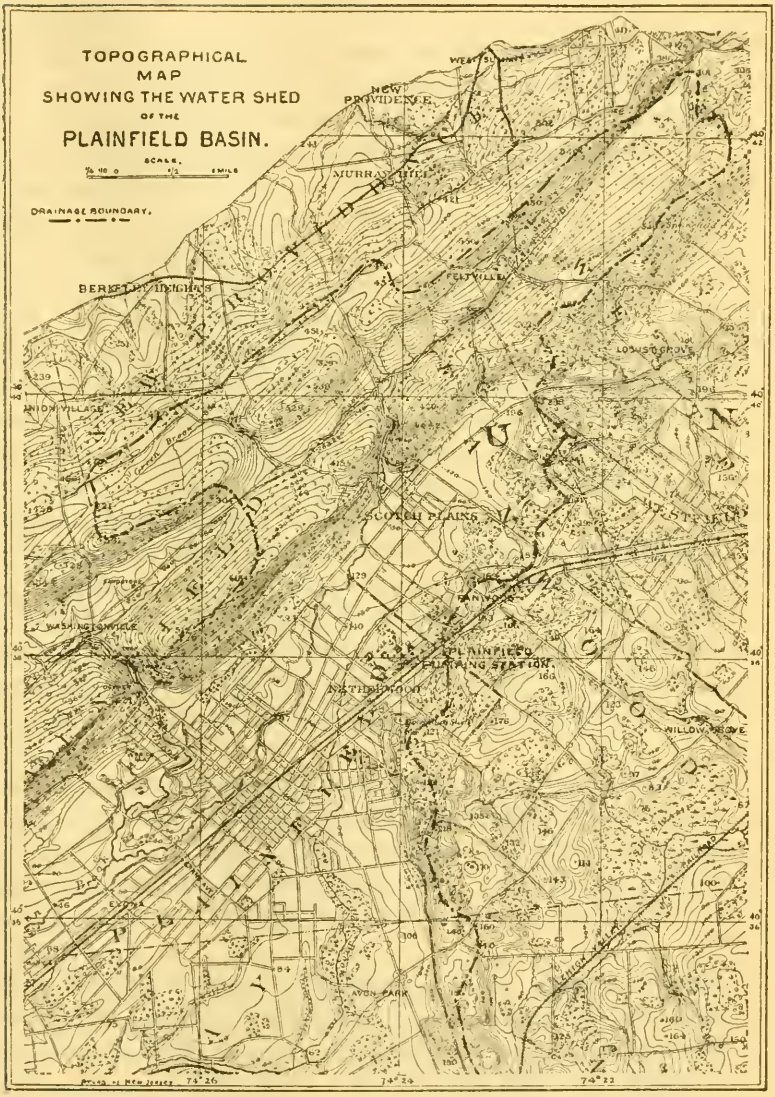
The soil consists mostly of sand, clay and gravel strata, rock not being encountered, except at considerable depths.

It has always been an easy matter to procure water in abundance for domestic use by driving pipe wells from 20 to 80 ft. deep at each residence, and attaching pumps directly thereto; and for fire supplies, sinking large brick curbs some 15 or 20 ft. into the gravel, gave an abundant flow. But obviously, with the increasing population and no sewerage system, individual wells became a source of danger to health, yet for nearly twenty years no definite result was accomplished, more than the mere organization of a private water company.

In 1890, active measures were taken and tests and examinations made, which finally resulted in the sinking of pipe wells on a plot of ground 1½ miles east of the center of the city in a soil where the upper clay stratum was some 30 ft. or more in thickness, underlaid by a very coarse water-bearing gravel. This spot was selected for its freedom from probable contamination on ground slightly higher than the city, which at the same time was convenient.

Several test wells were sunk at various points previous to the observations of the writer, and pumping tests made with a low-lift pump of a number of the main wells then driven, under the care of Mr. Rudolph Hering, M. Am. Soc. C. E. The quantity of water obtained from ten wells for periods of eight hours' daily consecutive pumping, during

PLATE XLIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 700.
TRIBUS ON DRIVEN WELLS AT PLAINFIELD, N. J.



two weeks of observation, was at the rate of from 2 000 000 to 2 125 000 galls. in 24 hours.

An inspection of Plate L will show the final arrangement of the wells, test wells, pumping plant in general, and details of the well tubes. The construction of the cast heads is such as to transform each water tube into practically an open well, giving atmospheric pressure free play rather than forcing its action through the earth, as in systems where but a single tube is used. The most distant well is 500 ft. from the pumps and shows in an interesting manner by the vacuum at the well head and increased vacuum at the pump the effect of long suction and friction in the main (see Table of Tests, page 376).

The 2-in. pipe test wells, marked A, B, C and D, on Plate XLVI, were observed daily by the writer, while resident engineer, during several months. They each had a simple balanced float-gauge and scale, which indicated the rise and fall of water-level. They were all very sensitive to draft on the main wells when pumping was going on, though the nearest was 200 ft. from the line of wells.

Comparison of these observations under the different conditions and seasons showed, among other things, that in about 1 900 ft. the underground water-level fell to the westward about 3 ft., or at about the same rate as the average surface of the ground. This evidenced conclusively that the flow of water was towards the city with a head sufficient to prevent any back flow of contaminated waters from the city.

In summary, the plant consists of twenty wells, 6 ins. in diameter, from 35 to 50 ft. in depth each, ranged in a double row on a strip of land 25 ft. wide and 1 000 ft. long, having in each a $4\frac{1}{2}$-in. open-end suction tube, connected with a wrought-iron main varying from 8 to 12 ins. in diameter. This main is in two sections, each 500 ft., connecting 10 wells.

Two compound, surface-condensing, duplex plunger pumps, Worthington make, one of 3 000 000 and one of 2 000 000 galls. daily capacity, and a boiler plant of sufficient power, with various essential small machines, are housed in a rough-stone building, slate roofed.

The water, drawn, as before stated, direct from the wells, is pumped into a wrought-iron stand pipe (situated near at hand) 25 ft. in diameter and 140 ft. high, through a 20-in. interior tube rising 5 ft. above the top. Two lower openings on this rising main, with valves operated

from the outside spiral staircase, afford opportunity for filling the stand pipe at lesser head if required.

The object of this interior tube, which was almost unique when erected, is threefold.

First.—By its fountain action, enforcing complete aeration.

Second.—Complete circulation.

Third.—To afford instant fire pressure, no matter what the elevation of water in the main tower. This is accomplished by opening a by-pass, not otherwise used, connecting the rising main and the distribution line, the city's supply being drawn regularly from the bottom of the stand pipe with pressure due to level of water in main tower.

From the stand pipe the Plainfield pipe system extends to the west, comprising some thirty miles of mains from 6 to 16 ins. in diameter, having fire hydrants spaced about 11, and valves 6 per mile.

Plate XLVII shows the "loop" or "double-feed" system adopted, dividing the city into districts, each practically complete in itself.

To the east lies a main running eleven miles to Elizabeth, supplying Fanwood, Westfield, Cranford, and the smaller places on the way.

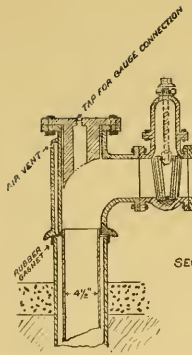
Blow-offs are placed at all low points and brook crossings. Hydrant pressures range from 40 to 75 lbs. per square inch according to locality, probably averaging 65 lbs. over the whole district.

From the plan of pipe lines (Plate XLVII) a general idea can be gained of the distribution of materials and the requirements for special castings, in estimating cost of construction.

Almost all street intersections are macadamized; about two miles have Telford pavement, and three miles Macadam. On other streets by following the adopted rule of laying 5 ft. from the curb, stone pavement and Macadam surfaces were avoided. Minimum cover, 3 ft.; surface of the streets to be replaced at once in good condition. No rock was encountered in trenching, but brooks were frequently crossed, usually below their beds, but in two instances as in sketches (Plate XLVIII).

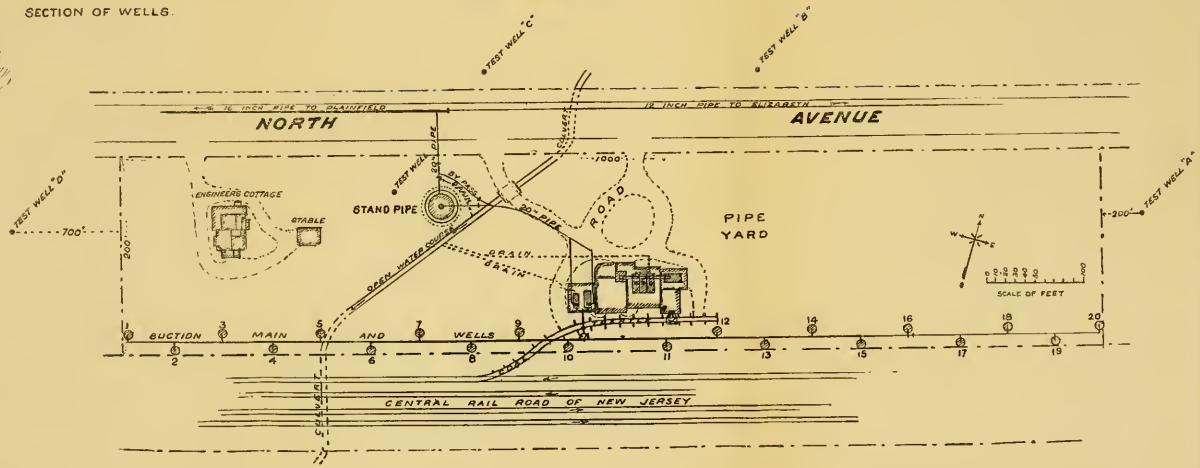
The cost of laying, including specials and 4-in. secondary valves for hydrants, but omitting the hydrants themselves, is given in the following table.

PLATE XLV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 700.
TRIBUS ON DRIVEN WELLS AT PLAINFIELD, N. J.



SECTION OF WELLS.

—PLAN OF PROPERTY—
—AND—
—DETAIL OF WELLS—
—PLAINFIELD WATER SUPPLY—
1891.



AVERAGE POUNDS OF MATERIAL PER LINEAR FOOT LAID.

	6-IN.	8-IN.	12-IN.	16-IN.
Pipe	30.00	44.00	76.00	124.00
Specials	0.70	0.90	1.80	3.10
Lead	0.85	1.05	1.84	2.05
Packing	0.01	0.02	0.02	0.03

COST PER LINEAR FOOT OF PIPE LAID.

	6-IN.	8-IN.	12-IN.	16-IN.
Pipe and specials	$0.394	$0.561	$0.965	$1.580
Lead and yarn	0.040	0.050	0.087	0.097
Valves and boxes	0.041	0.050	0.054	0.072
Tools and labor	0.124	0.165	0.177	0.260
Contractor and engineering	0.034	0.063	0.061	0.089
Total	$0.633	$0.889	$1.344	$2.098

But to return to the point of chief interest in these works, viz., the experiments and ultimate success of the wells. After the tests made by Mr. Hering and the partial completion of the works, various other tests were made with the permanent pumping plant. It was found that the wells on the westerly line yielded more abundantly than the easterly ones, under equally good conditions, and gave a lower vacuum for the same quantity pumped. This, taken with the observed difference of water level previously mentioned, tended to confirm the belief that the water came from an underground stream, flowing southwesterly, just skirting the eastern wells and flowing full past the western ones.

The tests were made with the large pumps, under both free discharge and full working head, singly and together, and drawing from the wells in groups of 5, 10, 15 and 20, using each combination of 5; also, by cutting off one by one until the smallest number that could be used was reached, then adding one by one in reverse order until the full series were again in use. Five wells were found to be the smallest number possible to use and run the pumps smoothly. Wells Nos. 6 to 10 gave the best results, while Nos. 16 to 20 furnished but little water. The best results were obtained for a full flow by using Nos. 1 to 15 inclusive.

The time covered by these tests ranged from 4 to 81 hours each, in several different months and with both low and high pump speeds.

From among them the following summary is given, to show approximately the relation between speed of pump, suction lift and friction loss between the observation points.

DATA.

Pump—3 000 000 galls. capacity, 24 hours.

Full speed—124 strokes per minute.

Delivery pressure—62½ lbs. per square inch.

Wells used—Nos. 1 to 10, inclusive.

Readings taken—30-minute intervals.

Length of test—24 hours, August 12th and 13th, 1891.

TIME.	Average number of strokes per minute.	Total United States gallons pumped.	Average suction lift in feet referred to level of upper pump valves.		
			Wells.		Air chamber in engine-room.
			No. 1.	No. 9.	
Hours.					
4	48.5	203 147	21.0	22.1	25.3
4	59.5	249 500	21.6	22.9	26.1
4	64.6	271 491	22.2	23.3	26.8
4	58.3	244 335	22.0	22.8	26.4
4	57.6	241 263	21.8	22.9	25.9
4	61.9	259 274	22.1	23.0	26.5
		SUMMARY AND AVERAGE OF ABOVE.			
24	58.4	1 469 010	21.78	22.83	26.16

The following test was made to determine the maximum flow, after the works had been in service for some five months.

DATA.

Engine 634—2 000 000 galls. per 24 hours.

" 635—3 000 000 " " "

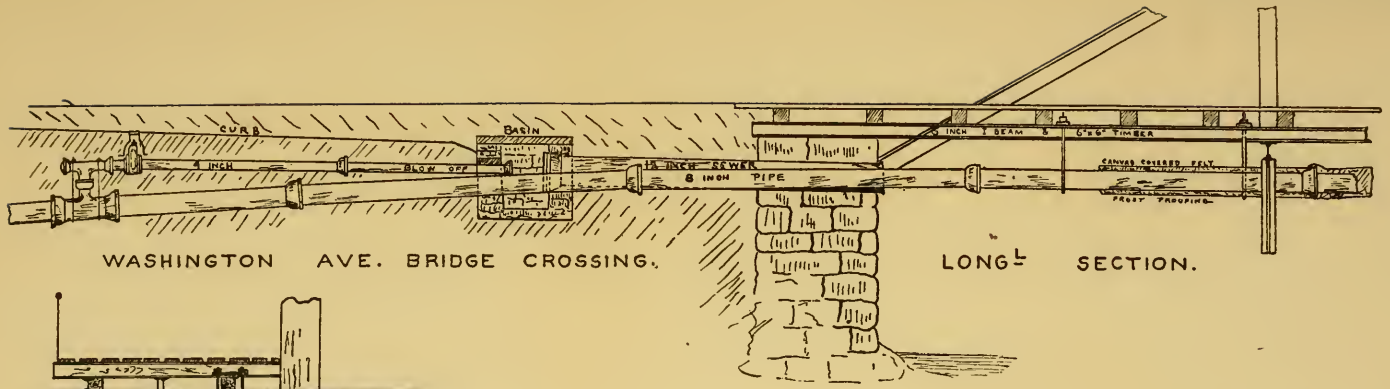
Full speed each—124 strokes per minute.

Delivery pressure—62½ lbs. per square inch.

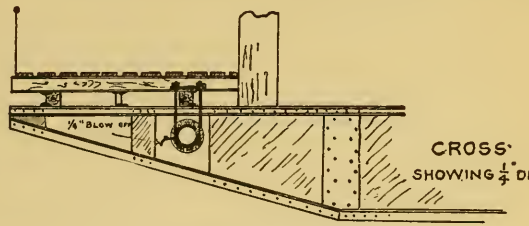
Readings taken at three-hour intervals, uniform running.

Test made by Amos Andrews, Engineer, March 8th to 11th, 1892, before lowering of pumps.

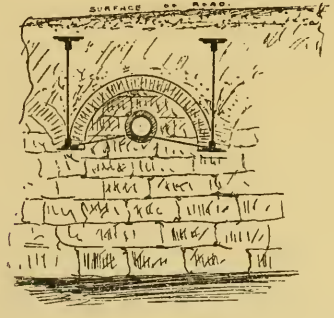
PLATE XLVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 700.
TRIBUS ON DRIVEN WELLS AT PLAINFIELD, N. J.



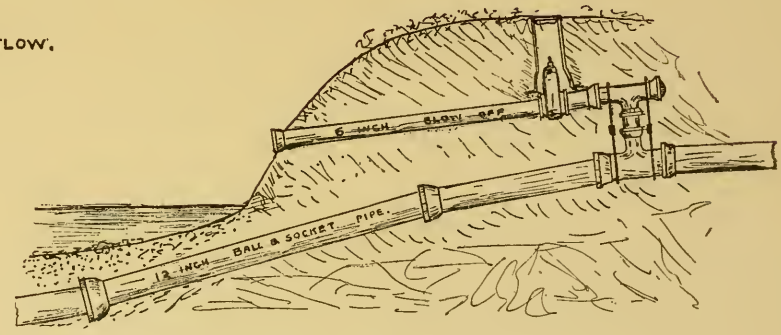
WASHINGTON AVE. BRIDGE CROSSING. LONG^L SECTION.



CROSS SECTION.
SHOWING 1/4" DRIP FOR WINTER FLOW.



DETAILS
OF
BROOK CROSSINGS.
PLAINFIELD-WATER SUPPLY.
1891.



SANFORD AVE. CROSSING.
12 INCH SPHERICAL JOINT PIPE.

SOMERSET ST. BRIDGE CROSSING.
8 INCH PIPE, SUPPORTED BY WROUGHT IRON SADDLES 3"x1/2", CARRIED ON FLANGES OF GIRDERS.

Time.	Average number of strokes per minute.		United States gallons pumped each 12 hours.			Average suction in feet. Readings from gauge in suction pipe.	Remarks.
Hours.	No. 634.	No. 635.	No. 634.	No. 635.	Total.	Air chamber.	
12	69 7	66.0	590 477	830 740	1 421 217	27.0	20 wells.
12	71.9	62.3	609 109	782 360	1 391 469	28.0	"
12	74.9	73.5	632 070	923 796	1 555 866	28.2	15 wells, 1 to 15.
12	73.4	77.7	621 813	977 340	1 599 153	28.4	" "
12	77.1	75.9	652 678	954 721	1 607 399	28.5	" "
12	82.1	76.4	695 305	960 586	1 655 891	28.6	" "

Summary of Above Table.

Hours.	Total United States gallons pumped.	Average suction lift in feet.	Wells in use.
24	2 812 686	27.5	20
24	3 155 019	28.3	15
24	3 263 290	28.55	15

During the long-continued dry weather of 1891, the water level became so low that difficulty arose with the extreme suction lift obtained, from 20 to 28 ft., according to rate of pumping, a fall of some 6 or 7 ft. since the earlier observations, so that in the summer of 1892 it was deemed best to lower the pumps, which was done to the depth of 8 ft. 1 in. below the former positions.

For the sake of a constant observation and record, a 3-in. open tube was driven from the engine-room, into the water-bearing gravel, and a permanent float gauge suspended in it, indicating by a balanced pointer on a scale of feet placed conveniently in the room. Although some 80 ft. from the nearest main well, therefore not showing the lowest level of the water at the wells when pumping, it does show the relative water-level under the same conditions and the daily and monthly range. When pumping, the average lowering of the gauge is about 8 ins., with an almost immediate return after stopping the pump.

Rainfalls need to be exceptionally heavy to make any marked showing in the water-level, and not much then inside of 24 hours. This seems to indicate that the water supply comes from a distance, but there is an insufficiency of data for determining this interesting point.

In these two years or more of operation, the wells have furnished daily, without difficulty or signs of falling away, the full demand of

from 200 000 galls. at the start to 1 700 000 galls. at the present time, apparently derived, as the early tests indicated, from the western fifteen of the twenty wells driven. The water itself has been of uniformly excellent quality, both for domestic and manufacturing purposes; so far, therefore, a decided success as an underground water supply.

DISCUSSION.

P. K. Yates, M. Am. Soc. C. E. (by letter).—I take this opportunity to give my experience in obtaining a supply for a system of water works by means of driven wells.

It was my privilege to design and construct a system of water works at Natchez, Miss., in 1887–89. The works were built on the franchise system, and it was especially designated that the supply should be taken from the Mississippi River.

The company was averse to taking the supply from the river, as it was found after several experiments that the water would not clarify itself after standing three days. So desirous was the company of securing pure water that would be acceptable to the people that it decided to drive wells, to determine whether a supply could be obtained in that way.

At that time the city of Memphis, some 300 miles up the river, and Bâton Rouge, 100 miles down the river, had obtained water from driven wells, and if water could be obtained in that way, it meant success for the company. At Vicksburg, about 100 miles up the river, a well had been put down in rather a crude manner without success. We knew, also, that the river water system at Vicksburg was not producing clear water, and the water was not generally used for drinking purposes.

Work was commenced on the wells, and after a few delays and trials we struck a water-bearing bed of sand 40 ft. thick and about 260 ft. below the surface. The sand was very coarse, and success was assured; the water flowed freely to the surface. Upon analysis it was found to be of a superior quality.

The manner of putting down the wells was, first, to put in a casing, thereby excluding all surface water, and then commence with pipe of

the desired size, and continue it nearly through the sand. The pipe had a notched cutting edge, and as it was made to revolve by machinery, it gradually wore its way down. Water was continually pumped down the center of the pipe, washing out all material in the center and driving it to the surface on the outside of the pipe. Hard material was sometimes encountered, when drills would be dropped down the wells and the hard material drilled away. The pipe would have to be pulled up occasionally, to put on a new cutting edge; hydraulic jacks, connected with powerful pumps, were used for this purpose. In one case the pipe was pulled apart, which necessitated starting a new hole.

After the pipe was in place, the strainer—about 30 ft. long—was dropped into it, and it was then pulled up 30 ft., the strainer remaining in the sand. A patented connection prevented the water getting into the pipe except through the strainer. The wells were a complete success. The pumps were lowered six ft. below the surface of the ground, so the water flowed freely to them.

The supply was more than sufficient for the capacity of the pumps, and continuous pumping had little effect in lowering the water in disconnected wells.

The works were finally completed, and the day set for testing the pumps. They threw the required six streams through 50 ft. of 2½-in. hose 100 ft. high, and the works were presented to the city for acceptance.

So prejudiced was the Board of Aldermen against this new method of obtaining water that it refused to accept works with this source of supply. The Board sent samples from the wells to three leading chemists of the South, together with samples from the river, and, although the results were favorable to the well water, it was only after every physician in the city had declared in favor of the latter that the Board decided to yield to the company. The experience of this company was no doubt similar to that of many others; it was asked to do something which it knew would not be beneficial to the company or to the city ; it was willing to incur the expense of determining whether water could be obtained from driven wells, and, having obtained a good supply, was not willing to abandon it. In my opinion this contest was perfectly legitimate.

It may be of interest to say that not one family in the city used river water to drink. Their drinking water was obtained from cisterns.

Many families have two or three cisterns which are all underground. They are about 16 ft. deep and 10 ft. in diameter, and hold about 9 000 galls. If the cisterns are well cleaned, the water caught only during the winter season, and care used in keeping the roofs clean, it would be hard to find purer water. In the majority of cases, however, there is only one cistern holding about 5 000 galls. of water, the conductor from the roof is connected with the cistern winter and summer, and no great care is taken in keeping the roofs clean. It can be readily seen in what condition the cistern would be after one year's use.

CHARLES B. BRUSH, M. Am. Soc. C. E.—As has been stated, these works were commenced by one company and finished by another. A mistake has been made in the statement as to the connection of Mr. Hering with the work; he was not employed by either company, but he was an expert appointed by the city to pass upon the work as it was constructed by the first company. When I was called in, the wells had been driven by F. W. Miller, of New York City, and had been found to be in a satisfactory condition. The foundations and the walls of the pump-house had been built, and the results of the investigation made at that time indicated that the better thing to do was to go on and finish the work as it had been accepted at that time.

The year in which the work was done was a wet one; there had been a considerable fall of rain and it was believed that the level of the water in the ground would remain practically constant, but the first year after the supply was put into use happened to be very dry. That indicated that this supply was not inexhaustible. The ground-water fell to a considerable depth below what was supposed to be its permanent level. That necessitated the lowering of the pumps some 8 ft., after the works had been in operation a couple of years. The supply has been very satisfactory. The quality is all that could be desired, and the quantity has held out remarkably well.

One of the important features, it seems to me, of this supply, is one that has been referred to by Mr. Tribus, that is, as to its aeration before its delivery. All waters taken from underground sources deteriorate rapidly if they are allowed to stand any length of time, and therefore it is desirable to aerate them as much as possible, and this was very successfully accomplished by forcing the water through an interior tube in the stand pipe. There have been, so far as I know, none of the difficulties from algæ and similar troubles that we often

find in underground supplies on Long Island and elsewhere. I think this is largely due to the fact that the water is kept so constantly in motion, and because there is an absence of the low elevations of water which we find on lower levels along the sea coast. The elevation of the water in the Plainfield wells, above tide water, is considerable.

Mr. TRIBUS.—The water level at rest is about 106 ft. above sea level.

Mr. BRUSH.—I think that accounts for the fact that we do not have the difficulties in these wells that we find in water taken from lower levels near tide water.

The literature on the subject of water taken from subterranean sources is not large, and I think that papers of this kind which describe actual experience, the difficulties and the methods of overcoming them are exceedingly important. The more carefully written papers we can obtain on this subject, the better it will be for the profession.

I submit the following data from memory, but let it be understood that these wells were all completed and accepted before I had anything to do with them. If you will turn to the diagram you will notice that 20 wells are shown on a line of pipe 1 000 ft. in length.

When the tests were being made as to the amount of water that could be obtained from the wells it was found that, as stated in the paper, the natural difference in level between test well *A* and test well *B* was about 3 ft. fall towards the City of Plainfield. Taking that as the normal condition of the water, at the time of pumping, the water was actually lowered in well No. 9 about 3.1 ft. In the test well in the house, which was not connected at all with any other wells and which was opposite the line between wells Nos. 10 and 11, the water in that test well was lowered about 14 ins. The water in test well *A* was lowered $8\frac{1}{2}$ ins.; in test well *B*, 8 ins.; in test well *C*, $7\frac{1}{2}$ ins; in test well *D*, 7 ins. You will notice that test well *A* is about 200 ft. from the eastern well of the series; test well *C* is about 300 ft. from well *B*, and test well *D* is about 800 ft. from well No. 1. Of course there were not a sufficient number of these test wells, at least when I was connected with the work, to determine exactly the nature and extent of the cone as completely as would be desirable to show it, but the data were sufficient to get an approximate idea of the extent and influence of the cone from the data which I have submitted to you. I would suggest that Mr. Tribus give the additional data asked for during the discussion.

A. S. TUTTLE, Jun. Am. Soc. C. E.—The water supply system of Brooklyn, N. Y., offers an illustration of driven wells operated under what may possibly be considered ideal conditions, since a large portion of the water-shed offers no ready surface drainage, and the sandy soil favors an underground storage, while the uniformly low elevations fail to afford site for other than shallow storage reservoirs, and these at great cost.

The total average daily consumption of water is about 75 000 000 galls., of which amount approximately one-third is supplied by driven wells located at five different points in the water-shed area. The driven well plants have all been constructed under contract to furnish the city with a given quantity of water; it can therefore be readily understood that no more wells were driven than were necessary to deliver the required amount. An unexpected interruption of a portion of the city supply of surface water about a year ago developed a peculiar case, in that two of the stations, after having been in operation for nearly eight years, proved their ability each to deliver 10 000 000 galls. daily instead of 5 000 000 galls., for which they were designed, and one of these plants has since continued to deliver the larger amount.

Mr. George H. Andrews, of the firm which built these plants, has advanced the theory that the continued use of driven wells opens the water-bearing strata, reduces the friction, and so affords more opportunity for the water to reach the wells.

A popular and widespread idea seems to exist that to sink wells anywhere on the southern slope of Long Island insures an almost unlimited supply of water of the finest quality, so that it may be well to state here that the common experience of those who have ventured in that field has proved that the locality in question fails to differ from others in this respect, and that the subsoil water has sought out its own channels, which must be located before a permanent and substantial yield can be expected.

Theory in regard to an underground supply in any locality is unsafe until it has been fully tested. In 1854, when Brooklyn was seeking advice as to the best and most economical method of securing water, an apparently exhaustive examination and report was made by J. S. Stoddard, C. E., on the subject of deriving a supply from wells. Mr. Stoddard concluded that the underground water was evenly dis-

tributed over the entire area, and that since the surface of the same on the southern side of the island showed a slope of 10 ft. per mile toward the sea, such a slope would be necessary to cause the flow through the underground strata; from this, as a basis, and assuming that the hydraulic grade line to the wells would have a uniform slope of 10 ft. per mile, the conclusion was reached that, if a well be sunk 10 ft. below tide and at a distance of one mile therefrom, it would receive salt water, and in like manner a well in the center of the island, where the width is 10 miles, could not penetrate more than 50 ft. below tide water, or it would become brackish. The fallacy of the theory is established by the fact that some of thedriven wells are operating at depths of 350 ft. below tide, and located less than 3 miles from salt water.

The deep wells usually have strainer joints inserted at various points in their length, so as to drain from the different water-bearing strata encountered, the surfaces of which may be at different depths all the way down to the bottom of the well.

In regard to the effect of pumping upon the underground water level, there was some time ago an article in *Engineering News*, by Prof. Trowbridge, which stated that numerous experiments made at one of the pumping stations showed that at 4 300 ft. from the wells the water was lowered 6 ins., at 2 300 ft. it was lowered 2 ft. 2 ins., and at 300 ft. it was lowered 4 ft. 8 ins.

At one of the stations, consisting of 150 2-in. wells driven to an average depth of 80 to 90 ft., the soil is naturally saturated to within a foot of the surface; but when pumping 5 000 000 galls. daily it is reduced at the wells about 12 ft., and when 10 000 000 galls. are pumped per day, it is still further reduced about 3 ft.

Mr. Andrews has stated that his experiments have fully proved that the water level on the southern side of Long Island is not affected outside of a radius of 1 000 ft. from the wells.

W. Kiersted, M. Am. Soc. C. E. (by letter).—There are, among others, one or two points brought out by the paper of Mr. Tribus, which I should like to discuss.

1. *The Resistances in the Suction Pipes.*—It is a well-known fact, whenever water is continuously drawn from a well or a system of wells, that a hydraulic slope of the ground-water is established, falling toward the well or center of draught, the profile of which in any vertical section is an irregularly curved surface. The head which induces and

maintains the flow into the well, regardless of that causing the natural flow through ground, is the difference of level between the natural surface of the ground-water and that in the well. This difference of level in a system of driven or bored wells, whether of the single or double tube kind, depends upon the vacuum produced by the pump in the suction pipes, and it naturally follows that the nearer the pumps and suction pipes are placed to the natural level of the ground-water, and the less the frictional resistance in the suction pipes, the greater, for any given vacuum, will be the static head causing the flow of water into the wells. Now, the resistance of flow through the voids of the water-bearing sands is too great to allow any of the available head to be consumed in unnecessary resistances in the suction pipes and in the pump-lift above the natural surface of the ground-water; therefore, in my opinion, it becomes essential to locate the pumps and suction pipes very near or even below the natural level of the ground-water at the time of construction, and to so proportion the size or sizes of the suction pipes that there will result but little frictional loss.

Although this method is not usually followed and may cause additional cost in the original construction, it will, I am sure, in nearly every case, be found economical in the end, since there is a tendency to a gradual depression, within limits, of the average natural water level within the area affected by draught, to a filling of the voids of the sand around the well strainers and to a general increase of frictional resistance to flow by corrosion of the pipes, which, if the pumps and pipes are not low enough in the ground, will cause a serious falling off of the water supply and a consequent extension of the well system. Besides there is good reason to anticipate annual fluctuations of the average ground-water level as the result of occasional droughts, just as was experienced in Plainfield when it was found necessary to lower the pumps about 9 ft. The inconvenience of putting engines and pumps deep in a pit can be largely overcome by using vertical pumping engines, which are far better adapted to this character of work than are the cheaper horizontal pumping engines.

The usual practice of gradually diminishing the diameter of the main suction pipe as it recedes from the pump, even to diameters of 8 and 6 ins., can, I believe, be improved by making this pipe large and with but slight changes, if any, in diameter; since it will thus reduce the labor of the pump in maintaining an effective vacuum, and

each well will discharge into it as into a conduit through which a comparatively large body of water is moving with a slow velocity.

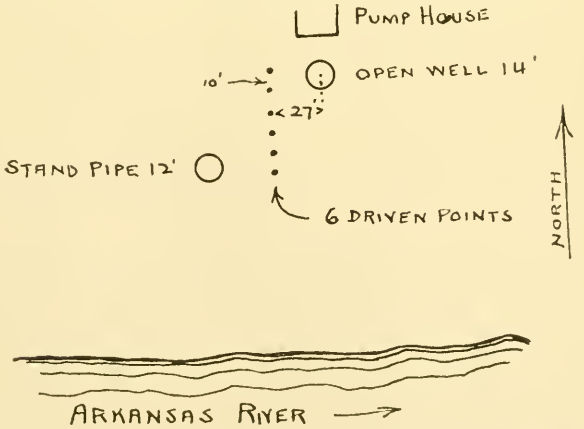
2. *The Distribution of Wells.*—The proper spacing of wells is largely affected by the coarseness and depth of the water-bearing sands. The coarser the sand, the greater the area influenced by the draught of each well, and the more widely should the wells of the system be separated. Nothing is to be gained by too thickly clumping the wells.

In an investigation and test, recently made, of a system of bored wells furnishing the water supply of a town of over 3 000 inhabitants, I found 17 wells, 3 to 6 ins. in diameter and 45 ft. deep within the circumference of a circle 125 ft. in diameter, ranged along a suction pipe 10 to 6 ins. in diameter. A pump test developed the fact that seven wells would furnish as much water as the 17. By means of test wells sunk to the same depth as the supply wells, I found the ground-water table to be depressed 0.2 to 0.3 ft. at a distance of 400 ft. from the wells of maximum draught, and 2 ft. at a distance of 95 ft. The water flowed very freely through a bed of clean coarse quartz sand and gravel about the size of a small pea. In this instance a much wider distribution of wells was necessary.

Tests made previous to the construction of a well system, similar to those made by Mr. Tribus, are essential before a proper arrangement of wells can be decided on.

It is a great advantage to have wells arranged symmetrically with reference to the main suction pipe, both with respect to distance and size of wells, since a more regular and evenly distributed flow of the ground-water follows.

H. V. Hinckley, M. Am. Soc. C. E. (by letter.)—If drive wells are large and perforated for a considerable length, there is, of course, a



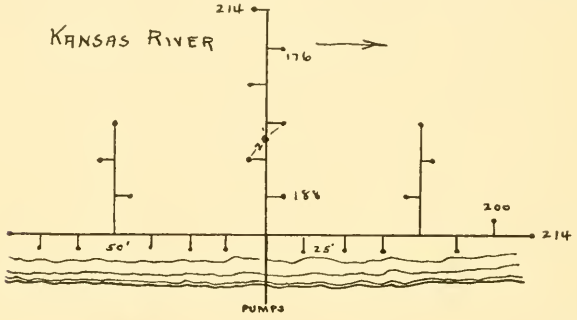
minimum distance at which they should be spaced. While the character of the water-bearing stratum must be a determining factor, I think 40 or 50 ft. may be roughly taken as the best distance.

At Great Bend, Kan., the superintendent of the city supply, attempted to increase it by adding six drive wells, 10 ft. apart. They are 72 ft. deep. They go through 30 ft. of clay, and have 12 ft. perforated at the bottom. Diameter, 4 ins. I recently tested these, and found that the maximum supply from one drive point was from 200 to 220 galls. per minute, while all six would only furnish 422 galls. per minute. In other words, Nos. 1 and 6 will furnish as much as all six.

The sketch on page 385 shows their location (scale, 100 ft. to 1 in.).

The tests for maximum supply at Plainfield show, for 15 wells, 148 galls. each per minute, and for 20 wells 97 galls. each per minute.

The new water supply at Topeka, Kan., includes 24 drive wells, 6 ins. diameter, 30 ft. deep, driven in the sand bed of the Kansas River to bed rock. Each well furnishes from 176 to 214 galls. per minute. No test has been made to determine the capacity of whole plant, but there is no doubt in my mind that after one hour's pumping one half the wells will give the same amount of water that the whole 24 will.



JOHN W. HILL, M. Am. Soc. C. E. (by letter).—Driven wells as a source of public water supply are largely used in the West. In many localities this is the only available source, and usually it is the least expensive method of procuring a supply of water sufficient for the necessities of a small community.

The city of Dayton, O., with a population of 80 000, draws its entire supply from a system of driven wells penetrating the gravel banks of Mad River from 35 to 40 ft.

The yield of driven wells depends upon the porosity, extent and depth of the pervious strata tapped. If the sand or gravel which constitutes the water-bearing strata is quite open, and free from soil or clay, and the area of the water-bearing bed is large, then such sources are more certain through the seasons than many of our smaller streams. There are many instances in the West of the complete failure of neighboring streams while the yield of driven wells has continued without material diminution during the dry period, although continuous pumping will usually show a gradual lowering of the level of the ground-

water. But as the water level is lowered, the grade of inflow to the foot of the well is increased, and the more remote portions of the pervious strata made tributary to the wells. As a rule, a system of driven wells will maintain the daily supply during drouth, excepting in those instances where the water field tapped is small in extent.

In locating driven wells near a city to be supplied with water, a good knowledge should always be had of the dip depth and covering of the water-bearing strata.

Instances are not rare where such wells have intercepted the subsoil drainage of the town to be supplied. Chemical analysis of water samples from test wells may not at once reveal the danger, while the steady pumpage will eventually train the seepage or leaching from cesspools and privy vaults into the water field.

The common belief that water from driven wells is always safe for drinking purposes is sometimes at fault. Typhoid fever has been traced to such wells, the germ having traveled from a neighboring infected privy vault through many feet of porous soil and finally finding an outlet to recommence its ravages through the driven wells.

In this connection it may be well to remark that pathological bacteria have been known to grow through the tubes of Pasteur filters. The germ becomes attached to the outer surface of the tube and by growth and fission will in due time appear on the inside of the tube and be found in the filtered water. Many of the pathological bacteria are as small as $\frac{1}{30000}$ of an inch in diameter, and can penetrate what would (excepting under the microscope) appear to be a solid substance. To remedy this evil, with Pasteur filter tubes, each time these are cleaned they should be boiled for 30 minutes, and then kept in an oven at 250° to 300° Fahr. for 30 minutes more, and sterilized so far as germs in the pores are concerned. Boiling alone cannot be depended on to kill all disease bacteria. In fact, absolute immunity from these can be had only with a degree of vigilance which it seems at all times almost impossible to exercise, or the exercise of which in the ordinary routine of life seems to be prohibitory.

In our homes it is not difficult to guard against the introduction of disease germs through our drinking water, but when we leave our homes and brave the drinking water of hotels and restaurants, then danger begins.

Who can say what kind of water is served in our public resorts, or, if the water be passed through a good filter, how are we to be assured that no disease germs or their spores are not locked up in the lump of ice with which the water is cooled? The filters used by our hotels and public dining-rooms are either incapable of removing all disease bacteria from a polluted drinking water, or the slovenly manner with which they are operated defeats this capacity (if it ever existed) in the ordinary sand and alum filter.

In the majority of cases water from driven wells is safe from a sanitary point of view. Surface drainage can reach wells only after an amount of natural filtration which will remove all the organic matter in suspension and the greater part of that in solution, and generally surface water is not to be regarded as dangerous after it has passed vertically through 35 to 100 ft. of drift above the lower ends of driven wells.

If, as sometimes happens in selecting sites for driven wells, the material overlying the water-bearing strata is very loose or consists of loam or gravel, surface water may find its way along the casing pipe to the foot of the well and introduce an element of danger, provided such surface water contains pathological germs or organic matter.

The late Mr. J. C. Hoadly undertook to develop an equation for the yield of driven wells from certain known data, but the extent of the water-bearing strata (generally unknown), the coarseness or fineness of the pervious material holding the water, and the relative amount of soluble soil in the interstices of the sand and gravel, all have such a strong influence on the yield of such wells that no rule can be capable of general application. If such wells are driven and tested at the end of the dry season and are found after several days' uninterrupted pumping to maintain a certain minimum water level (usually shown by a vacuum gauge on the suction main) the daily safe capacity for the wells can be stated with tolerable exactness.

A knowledge of the several borings for a system of driven wells is very valuable as affording the means of determining the probable capacity of such wells, provided the water-bearing strata tapped is large and capable of maintaining a constant supply equal to the requirements of the community to be served. Thus, if the water field is prospected and known to be of large extent, and the porous strata is open, as at Plainfield, a system of driven wells can be depended on for a large daily supply, but if the pervious strata consists of fine sand, while it may be very large in area, the water will come to the wells at such a slow rate as to make the yield or capacity per well very small, notwithstanding the field tapped may contain water sufficient for a large daily consumption. In such cases it is advisable to drive many wells of small diameter over a large area.

I have projected and tested quite a number of systems of driven wells for public water supply, and have found the conditions affecting the water yield to vary so much that a test of each system by pumping continuously for several days, and a careful survey of the water field, seem to be absolutely essential to an estimate of the probable capacity of such wells.

R. Willard Ware, M. Am. Soc. C. E.—I have been much interested in the subject under consideration, and am very glad that Mr. Tribus has presented the case so fully and clearly and given statistics and

data that are exceedingly valuable, and that probably could not be obtained from any other source. I think we can fully appreciate the value of the paper prepared by Mr. Tribus.

As a resident of Plainfield, I am interested in its public improvements, and particularly in its water supply. We were without city water works till within about two years. We now have a good supply of the best water known in the State for domestic use, and a pressure of 70 lbs., which may be increased to the full pump pressure for fire protection.

Judging from the numerous tests made and the amount pumped during the past year, together with the comparative heights of water in test wells, it is believed there is an ample supply of water, not only for Plainfield, but a surplus to spare for the neighboring cities and boroughs between Plainfield and Elizabeth.

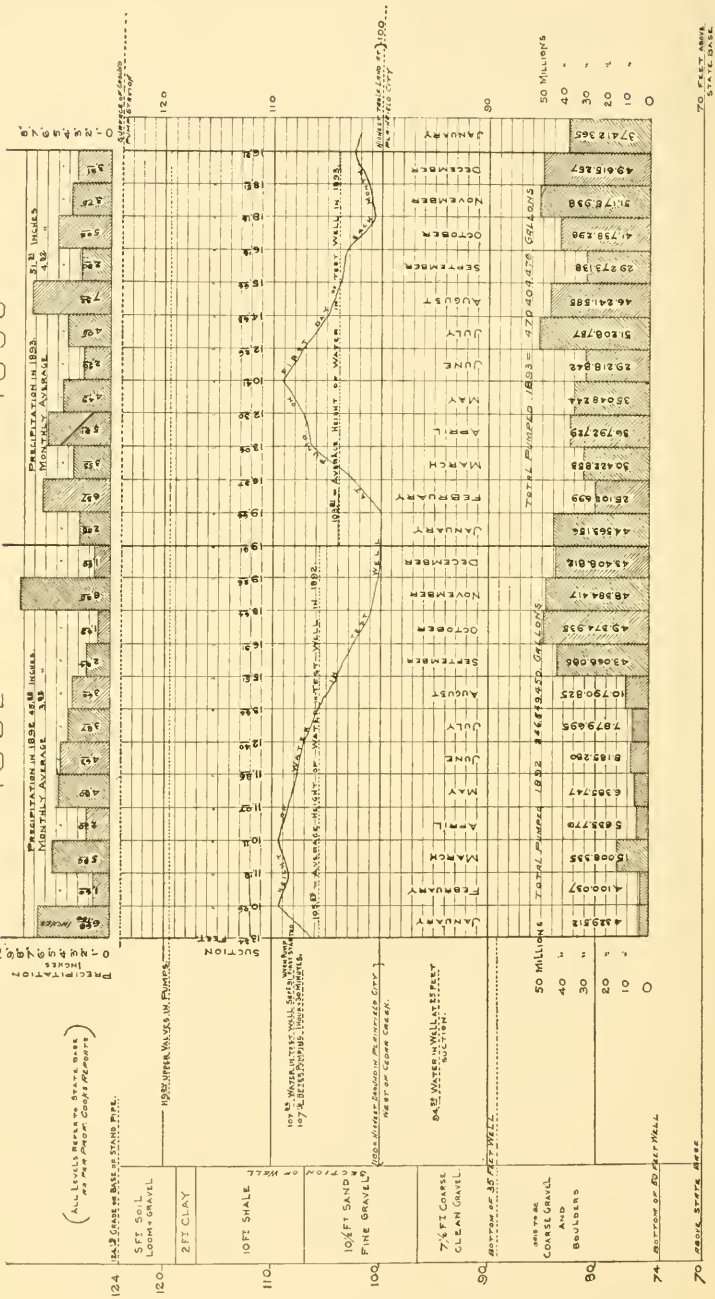
Should the water company at any time realize that the supply at the pumping station becomes inadequate to the demands, auxiliary wells may be put down at some distance from the station and operated inexpensively by the power plant that is now provided for the present station.

It is therefore believed that Plainfield's future requirements are not jeopardized by the Water Supply Company extending its water mains to supply our neighbors at the present time. Some questions have arisen in my own mind for the solution of which I have prepared a chart of the facts obtained regarding the pumping station; showing section of fluctuations of height of water in test well in pump house; suction lift of pumps; amount of water pumped monthly and annually; also, the precipitation monthly and annually in 1892–93 (see page 390).

The levels given on chart or referred to are all based upon the State datum, as per Prof. Cook's reports and contour maps of New Jersey.

Samples of material through which wells were driven show, first, 5 ft. to be soil, loam and subsoil; then 2 ft. of clay; below that 10 ft. shale; then $10\frac{1}{2}$ ft. sand and fine gravel; then $7\frac{1}{2}$ ft. coarse gravel, from which all sand appears to have been washed out—this being a good water-bearing stratum. This takes us to the depth of the 35-ft. wells. Samples below this depth not obtained (said to be coarse gravel and boulders).

Elevation of surface of ground at pump station	124
" " bottom of 35-ft. well	89
" " upper valves in pump engines	119.50
" " base of stand pipe	124.19
" " top of stand pipe	264 +
" " highest ground in Plainfield west of Cedar Creek	100


1892
1893
PRECIPITATION IN 1892 45.88 INCHES
MONTHLY AVERAGE 3.82
PRECIPITATION IN 1893 51.21 INCHES
MONTHLY AVERAGE 4.22
PRECIPITATION INCHES
SUCTION FEET
TOTAL PUMPED 1892 346,549,450 GALLONS
TOTAL PUMPED 1893 = 470,404,470 GALLONS
50 MILLIONS
70 FEET ABOVE STATE BASE
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
5 FT SOIL LOOM + GRAVEL
2 FT CLAY
10 FT SHALE
10½ FT SAND FINE GRAVEL
7½ FT COARSE CLEAN GRAVEL
COARSE GRAVEL AND BOULDERS
BOTTOM OF 35 FEET WELL
BOTTOM OF 50 FEET WELL
ABOVE STATE BASE

As the pumps now stand 144.69 ft. lower than top of the water tower they work against about 62½ lbs. static pressure when the tower is filled.

It has been ascertained by careful observation, and is a matter of record interesting to notice in this connection, that the coal (fuel) consumed in pumping is 1 lb. for 300 galls. pumped into the system under a constant pressure of 62½ lbs. per square inch, the net cost of coal being $2 95 per long ton (2 240 lbs.).

One ton of coal pumps 672 000 galls., or a cost for fuel of $4 39 per 1 000 000 galls. This is the cost at the present cost of coal; it has been purchased at a lower price, so that the expense for fuel was, until quite recently, $3 95 per 1 000 000 galls.

I take pleasure in acknowledging the courteous treatment received from Mr. Amos Andrews, Superintendent at the works; Messrs. Dunham & Gavett, City Engineers of Plainfield; and Mr. John Neagle, Observer and Reporter for the State Weather Bureau.

L. L. Tribus, Assoc. M. Am. Soc. C. E.—In the course of oral discussion on this paper, a desire for further information from the note book of the author was desired.

I take pleasure, therefore, in adding certain more detailed notes, and a few not at first included or contemplated.

Among the elevations should be noted the following (above sea level):

Streets at center of the city	100
Surface at pumping station	122
Water level at pumping station	106
Upper valves of pump (old position)	126
" " " (new ")	118
Flow line of stand pipe	262

Examination of existing scattered private wells by Mr. Hering indicated the advisability of running the wells in a northwesterly line from the pump-house site, intercepting the underground flow, which seemed to be in a generally southwesterly direction. But other reasons of convenience and policy determined the placing of the wells in a southwesterly direction instead of northwesterly, with the result mentioned in the paper, of but a partial intercepting of the flow.

Referring to the tests of the wells, it was found that when they were used in small groups, they yielded more water relatively than when all, or nearly all, were under suction, except that the increased number of wells gave a decreased suction lift.

But for the latter important fact, the results would imply that a lesser number of wells than twenty, more widely separated, would have produced an equal amount of water; in that case, however, the greater friction in longer suction lines would probably have destroyed such advantage.

Mr. Kiersted and Mr. Hinckley have both contributed interesting items on this point, and the following record of experiments at Plainfield still further illustrates it.

Experiment Number.	Wells in Use.	Rate of Flow in Gallons per 24 Hours.	Total Suction Lift in Feet.	Average Gallons per 24 Hours per Well.	Wells' Number.	Remarks.
1	10	1 470 000	26.2	147 000	1 to 10	August 12th–13th, 1891, 24 hours' run.
2	5	1 131 190	27.0	226 240	6 to 10	October 13th–14th, 1891, 20 minutes' pumping on each combination of wells in Experiments Nos. 2 to 24.
3	6	1 172 000	26.7	195 330	6 to 11	
4	7	1 283 600	26.7	183 370	6 to 12	
5	8	1 158 300	27.2	144 800	6 to 10 + 18 to 20	
6	8	1 361 500	26.6	170 190	6 to 13	
7	9	1 129 500	27.3	125 500	East Group.	Varying combinations.
8	9	1 327 620	26.6	147 510	6 to 14	
9	10	1 137 970	26.8	113 790	6 to 10 + 16 to 20	
10	10	1 254 810	27.3	125 480	11 to 20	
11	10	1 400 440	25.5	140 040	6 to 15	
12	10	1 615 500	28.4	161 550	1 to 10	
13	11	1 151 510	26.3	104 680	6 to 10 + 15 to 20	
14	11	1 329 320	26.2	120 840	6 to 16	
15	12	1 185 380	26.0	98 780	6 to 10 + 14 to 20	
16	12	1 417 380	25.5	118 110	6 to 17	
17	13	1 254 810	25.6	96 520	6 to 18	
18	13	1 259 890	26.2	96 920	6 to 10 + 13 to 20	
19	14	1 219 250	25.3	87 090	6 to 10 + 12 to 20	
20	14	1 437 220	26.2	102 670	6 to 19	
21	15	1 327 620	25.1	88 510	6 to 20	
22	15	1 442 770	25.6	96 180	1 to 15	
23	16	1 700 170	26.4	106 250	1 to 16	
24	20	1 388 590	25.0	69 430	1 to 20	
25	10	2 040 390	28.4	204 040	1 to 10	October 23d, 1891, 20 minutes' pumping on each group. Experiments Nos. 25 to 35.
26	10	1 763 980	28.0	176 400	1 to 5 + 11 to 15	
27	10	1 432 290	28.8	143 230	1 to 5 + 16 to 20	
28	10	2 085 620	27.9	208 560	6 to 15	
29	10	1 809 220	27.0	180 920	6 to 10 + 16 to 20	
30	10	1 427 270	27.9	142 727	11 to 20	
31	15	2 377 110	26.9	158 470	1 to 15	
32	15	1 789 113	27.6	119 270	1 to 5 + 11 to 20	
33	15	1 879 574	27.3	125 305	1 to 10 + 16 to 20	
34	15	1 774 030	28.4	118 270	6 to 20	
35	20	2 000 200	27.9	100 010	1 to 20	
36	15	3 210 000	28.4	214 000	1 to 15	March 8th, 1892.
37	20	2 800 000	27.5	140 000	1 to 20	

It will be noted, perhaps, that in the preceding record but rarely did wells Nos. 1 to 5 appear in use, the reason therefor being, that earlier tests had developed their satisfactory conditions, as also was the case with Nos. 6 to 10; but these latter were used to furnish a sufficient volume of water to work on while testing wells Nos. 11 to 20. Nos. 17 to 20 were found to be almost worthless.

The experiments showed that after continued operations the wells yielded relatively more water in the later tests than in the earlier without a corresponding increase in suction lift, a fact due, no doubt, to the opening up of the underground channels leading to the wells.

Regarding the range of susceptibility of water level to pumping, tests of such level shown in Figs. 1 and 2 fairly illustrate the average

results under regular operation, but observations were not taken from enough points to demonstrate with scientific precision the radius of effect.

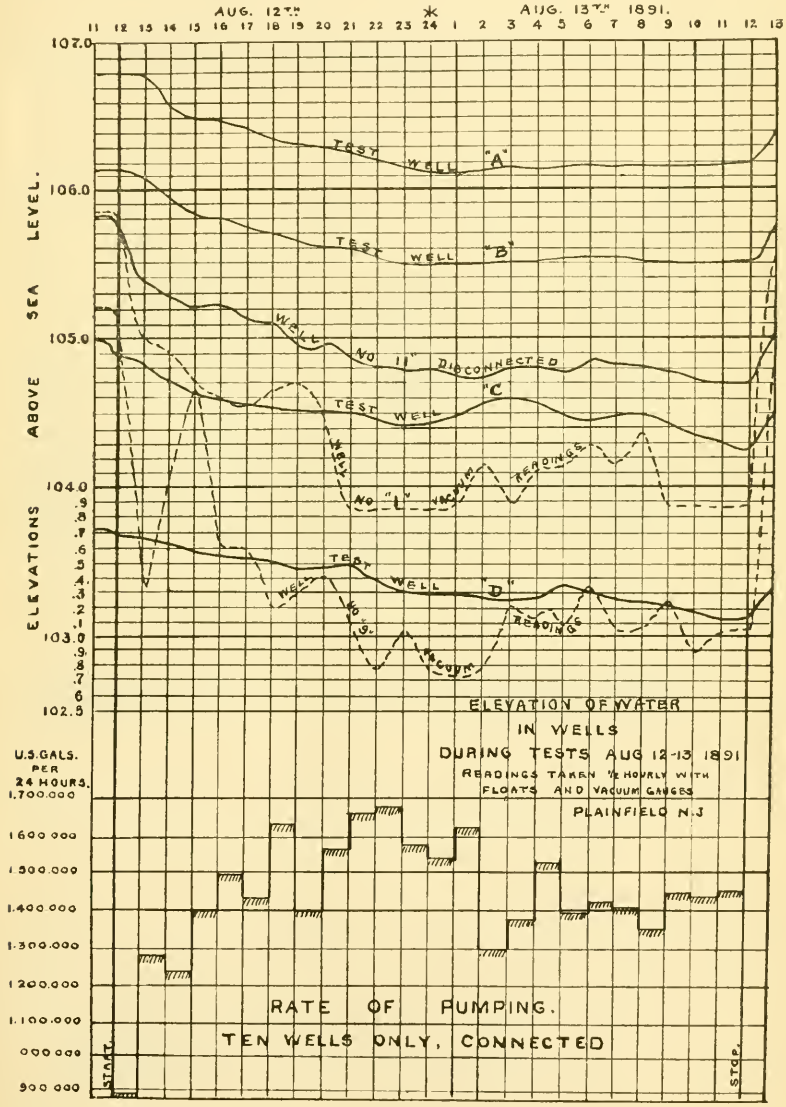


FIG. 1.

It is to be regretted, perhaps, that corporations are not willing to spend money for such scientific experiments after sufficient for all

practical purposes have been made, as definitely stated in the paper. Speaking from some experience, I have never discovered such willingness or deemed it proper to ask appropriations for such an object.

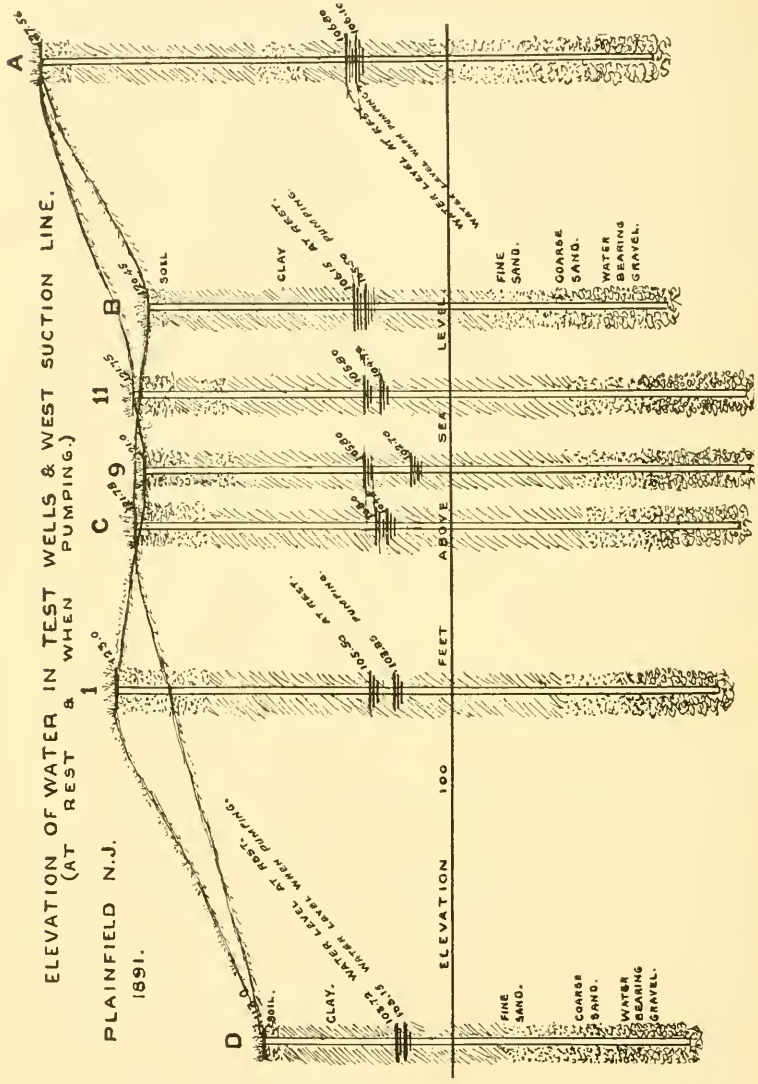


Fig. 2.

Fig. 3 shows the 72 hours' test graphically, giving, in addition, the water level in the house test well, with its lowering of some 3 ft.

at the maximum draft. As the test was made only with reference to quantity that could be pumped, no observations were made of the water level in test wells *A*, *B*, *C* or *D*.

An item perhaps of interest in the practical operation of the works came to my attention about two months after the formal opening of the system.

It demonstrated very nicely the adequacy of the mains and efficiency of fire service.

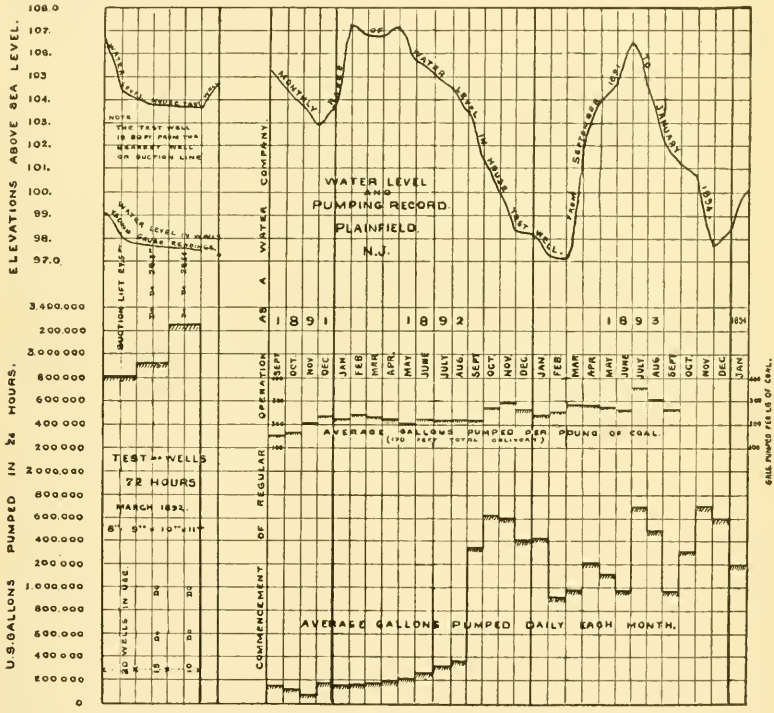


FIG. 3.

At 17.45 o'clock on December 3d, 1891, a fire broke out in a large lumber and coal yard in the center of the city (elevation about 100 ft.). There being at the time no hydrant contract with the city, only such as could be of service to the company for blow-offs had been placed in the system, though branches and valves were placed for the full proposed number.

Four of these were within an average distance of 500 ft. of this fire, and were at once placed at the disposal of the fire department. One had four $2\frac{1}{2}$-in. nozzles supplied by a 6-in. branch; the others, two nozzles each from 4-in. hubs.

Within a few minutes seven streams were in play, three from the 4-way; some from the hydrants direct ; others reinforced by steamers.

Seventy pounds per square inch was the static pressure at the hydrants, when nozzles were closed.

Until 8.45 o'clock on December 5th, a period of 39 hours, there was furnished the equivalent of 160 hours' continuous service through 500 ft. of hose and a $1\frac{1}{8}$-in. nozzle, using a total of 1 130 000 galls. of water, or an average of about 118 galls. per stream per minute.

At no time were the pressures perceptibly disturbed in any other part of the city. The stand pipe was kept nearly full all the time, and frequently the pumps had to slow down, even during the maximum draft, to prevent overflowing the tower.

As regards the source of this water supply, it seems sometimes that the water must come from a distance, rather than from purely local infiltration within the indicated water-shed, which seems rather more limited than the abundance of water suggests ; but at present, the question is not a serious albeit an interesting one, as the graphic record (Fig. 3) of the $2\frac{1}{2}$ years' operation shows.

It is a pity that in so many instances the same engineer who was connected with the establishment of driven well systems is not familiar with their later operation. We are thereby deprived of the benefit resulting from comparative records under early and later conditions, arranged on the same basis, by which only can such records be made of real scientific value.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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701.

(Vol. XXXI.—April, 1894.)

CABLEWAYS.

By Spencer Miller, Assoc. Am. Soc. C. E.

Read January 17th, 1894.

The cableway may be defined as a hoisting and conveying device employing a suspended cable as a trackway, and to such the author will limit this paper. Cableways are sometimes called "cable derricks" or "cable hoists." What are known as "wire rope tramways," "wireways," "ropeways," whether inclined or otherwise, have no capacity for hoisting, and are limited to the sole function of conveying.

While wire rope tramways date back to the early part of this century, the cableway finds its birth practically in an inclined hoisting and conveying device invented about 1860, by Charles Shuman, and the device as he designed it at that time is still in extensive use in the slate quarries of Pennsylvania and Vermont.

These cableways consist of a winding engine with one drum, a suspended cable, a cable carriage traveling on the cable track-way, a fall block adapted to rise and fall from the cable carriage, and a hoisting rope operating the same. Fig. 1 shows the early form of incline

cableway with the cable carriage in gate. The angle of incline of the cable is about 23° from the horizontal (see Fig. 2), measuring to the air line from top of "*A*" frame to tail anchorage. The hoist rope

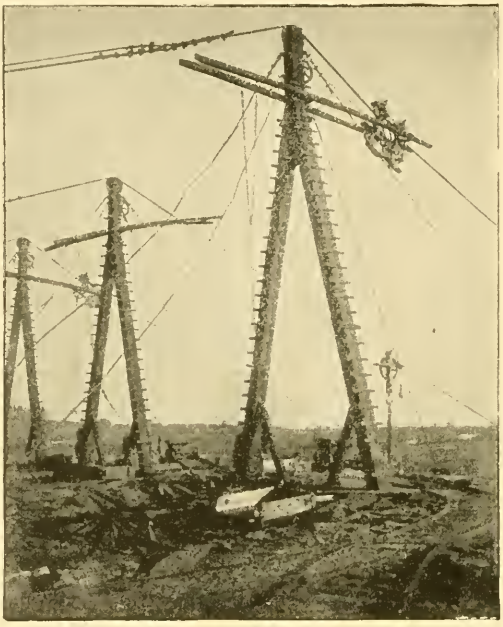


Fig. 1.

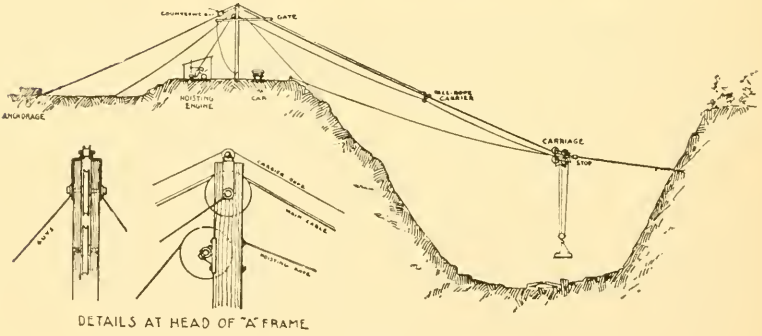


Fig. 2.

is multiplied three times from the carriage down to the load, the result being that at the minimum angle of 23° the resistance to drawing the

carriage up the incline is greater than the effort required to hoist the load. Therefore the carriage, resting against a stop on the cable, will not begin to ascend until the fall block reaches and comes into contact with it. Means are provided that the fall block may collide with the carriage without injury to any of the parts.

After unloading the fall block is raised to the carriage, the carriage drawn up the incline sufficiently to release the gate, then the hoist rope is paid out, the carriage descends by gravity down the incline cable until it reaches a stop on the cable, and, continuing to pay out hoisting rope, the fall block descends to the pit.

The wheel shown in the head of the "*A*" frame, or tower (Fig. 1), is one of the early mistakes, as illustrated by the repaired cable to the right side of the head of the frame. The best practice now is to employ an oak saddle, grooved with an arc of large radius to correspond with the stiffness of the rope (see Fig. 3).

The cableways shown are introduced about 30 ft. apart. There are probably 300 of them, known as "Blondins," now in existence. The load usually carried is from 3 to 5 tons. Sometimes single masts have been used in place of "*A*" frames. They, of course, require more guying for their stability.

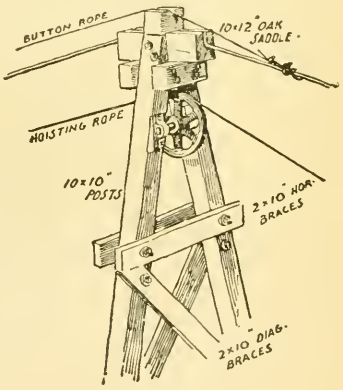


FIG. 3.

As the span of these cableways becomes greater, the necessity of supporting the hoisting rope between the tower and carriage becomes manifest. Fig. 2 shows the earliest device of its kind. This fall-rope carrier follows the carriage down the incline, drawing out a small manilla rope, permitting the carrier to go down only half way. At the other end of the manilla rope a counterweight is secured, so that on the return of the carriage and fall-rope carrier, this piece of manilla rope could be drawn out of the way from entangling with the other ropes. This carrier is limited to short spans ; not more than two have ever been used, and it is not to be considered as a solution of the difficulty. Much experimenting has been done to improve it.

Horizontal cableways of short span were used in the construction

of the piers of the St. Louis bridge, and a patent, dated January 24th, 1871, was granted to Henry Flad and James B. Eads, the former a Past President and the latter a Past Vice-President of this Society. The span was short, and no fall-rope carriers were required.

About 1884, Milo W. Locke, a contractor, designed an incline cableway for building the bridge piers of the Baltimore and Ohio bridge over the Brandywine, near Wilmington, Del. He afterwards took this same plant, making it horizontal by the addition of an endless rope for the horizontal motion of the carriage, and installed it in a quarry near by, of which he became manager. On this plant he used the chain-connected fall-rope carriers which consisted of a series of blocks with 8-in. or 10-in. wheels, to run on the main cable, spaced about every 50 ft., connected with ½-in. chains. These heavy and cumbersome fall-rope carriers were the source of endless annoyance. The span of the original cableway was about 750 ft. Mr. Locke received patents about 1884 for his invention. The cableways employed at the Sodom Dam and Tilly Foster Mine were modeled after this.

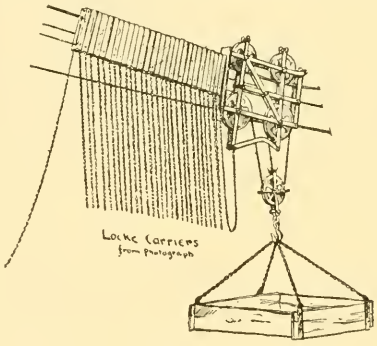


FIG. 4.

At the Sodom Dam 32 chain-connected carriers (Fig. 4) about 20 ft. apart were used (the span being 667 ft.). The hoisting rope does not need supporting oftener than every 100 ft., but with the chain-connected carriers, the chains themselves must be supported; hence the great number of carriers employed.

The weight of chain and carriers used on this plant was about two tons. The chains swing about and get entangled in the fall block, and with each other, they limit the speed, and wear the cable.

This Sodom Dam cableway was not designed by an engineer, and possibly to this fact is traceable the deplorable accident in October, 1889, when the main cable broke, resulting in the death of a laborer. The reason it broke was because it was strained up too tight, allowing too small a sag or deflection. This cableway was an assemblage of parts picked up here and there, without regard to efficiency. The engine was built in New York, the carriage in Pennsylvania, the cables in New Jersey. The main cable broke with a sag of 15 ft., and a load of 6 tons. With a span of 667 ft,, sag 15 ft., and load of 6 tons, the strain on a 2-in. cable is about 80 tons. This cable, when new, tested and broke at 132 tons. It is not surprising, therefore, that a cable used constantly for nearly two years, should break at 80 tons, when its ultimate strength, new, was only 132 tons, showing a factor of safety less than two. Cableways designed by the writer employ a factor of safety from four to six.

The towers on this plant should have been high enough to allow a sag of 33 ft. However, the plant was repaired, the towers were raised, so as to allow a sag of 22 ft., and the dam was completed with no further accident. The Sodom Dam was fully described before the Society on April 5th, 1893, by Walter McCullough, Assoc. M. Am. Soc. C. E.

When the Tilly Foster mines were begun on the open-pit plan, two horizontal cableways were employed, with chain-connected fall-rope carriers, and, as soon as the depth permitted, these were changed to incline cableways, employing the same fall-rope carriers. This is the first instance where cableways were used for "open pit mining," and to Mr. Clinton Stephens, contractor, belongs great credit for this novel installation; 329 000 cu. yds. of rock were thus moved to uncover 600 000 tons of ore.

One great advantage of using a cableway lies in the employment of a shallow skip, which, as will be observed, is easily filled by hand, with no preliminary work after a blast. At least two tons per man per day more may be filled into a skip 18 ins. high than into a car 36 ins. high.

Long and high-speed cableways were not practicable with the chain-connected fall-rope carriers. Fig 5 shows the first departure therefrom, and represents the beginning of a development in the line of improved fall-rope carriers. As will be observed, an auxiliary rope, about $\frac{5}{8}$ in. diameter, is suspended above the main cable, held in a par-

allel position to the main cable, by passing under wheels in the cable carriage.

On this rope a series of buttons are secured, whose diameter increases with the distance from the head tower. Slots in the head of the carriers, corresponding to the diameter of the buttons, allow each of the carriers in passing down the incline to be stopped at its proper button. These carriers have small wheels to roll on the auxiliary or button rope. Thus, the heavy cumbersome chains are dispensed with, and these fall-rope carriers, spaced by buttons, and weighing about 100 lbs., answer all the requirements of chain-connected carriers, weighing, with the chain, 1 000 lbs.

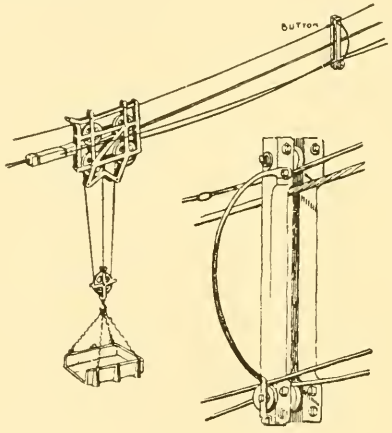


Fig. 5.

The button stop fall-rope carrier was next applied to a horizontal cableway of 855-ft. span, at Ogden, N. J. Means had to be provided for drawing the fall-rope carrier out with the carriage, as gravity was not to be depended upon as in the last case. To this end, two horns were placed upon the carriage, which served the twofold purpose of lifting the carriers bodily from the cable, thus dispensing with wheels on which the carrier might run on the main cable, and also, on these horns the carriers were taken when distributing them along the cable.

The original carrier was of wood and iron. The development of a few years has brought the carriage and carriers up to a standard form, as shown in Fig. 6. The cable carriage is usually built with two main

cable wheels, but in some instances three wheels have been used, notably in the Point Pleasant plant, 1 505 ft. span, which will be described later. The various ropes are indicated in the illustration. The horn in front of the carriage picks up the carriers as it passes toward the engine and also carries them out as the carriage recedes; the buttons on the button rope take the carriers from the horn and leave them spaced along the main cable at proper intervals for supporting

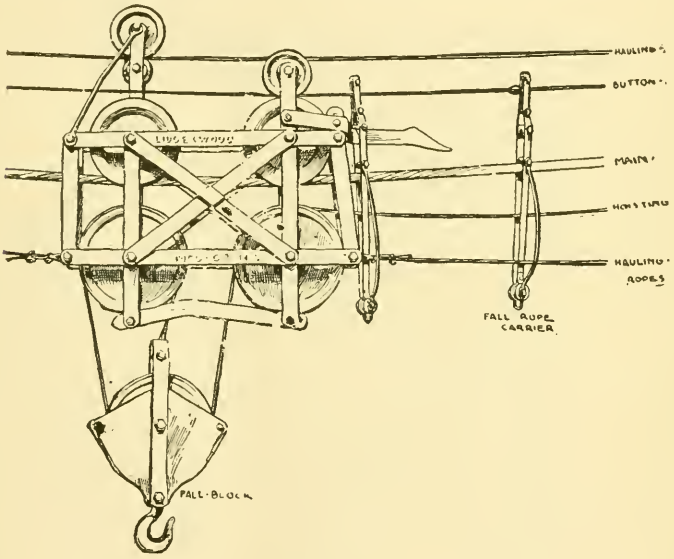


FIG. 6.

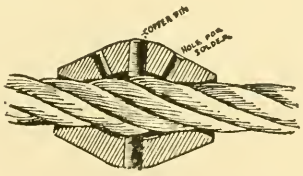


FIG. 7.

the hoisting rope. These buttons (Fig. 7) increase in size, receding from the head tower, as also do the corresponding slots in the head or top of the carrier.

Fig. 8 illustrates an engine with double cylinders with cranks connected at an angle of 90°, and is fitted with reversible link motion. The drums are 54 ins. diameter, regular friction type, one to carry the

hoisting rope, and the other is turned with a curved surface, as shown, and carries the endless rope. The endless rope is wrapped around the drum five or more times, enough to secure sufficient friction to keep it

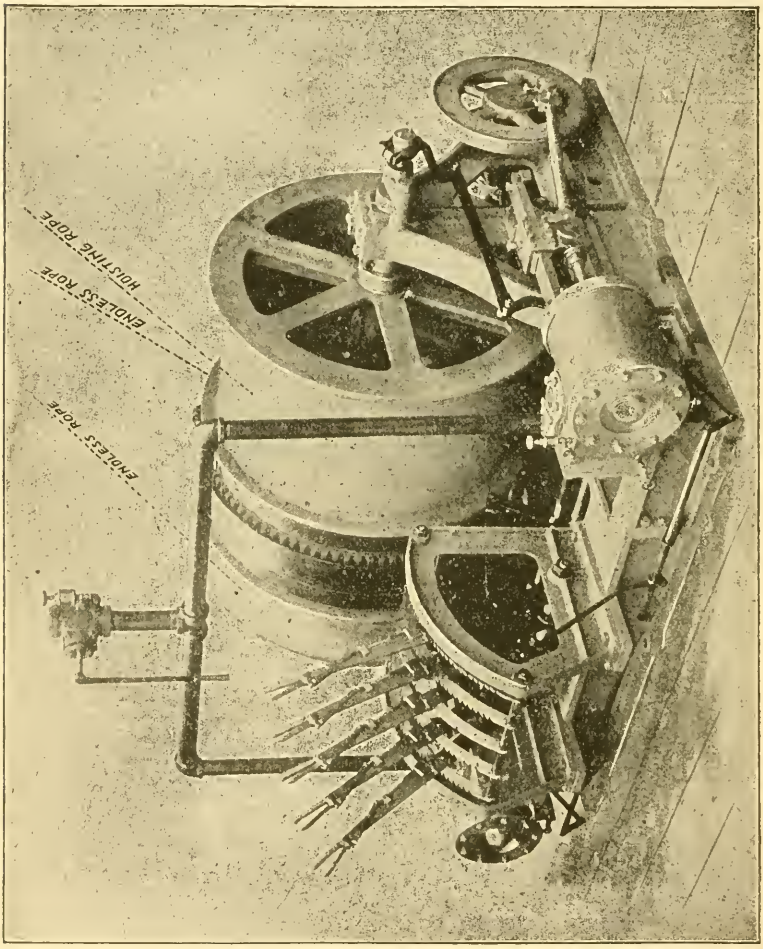


Fig. 8.

from slipping in the opposite direction to that in which the drum is turning, and the ends are passed over the sheave wheel on the towers and made fast to the front and rear wheels of the cable carriage.

The hoisting drum is independent of the other, and, being of the

same diameter, winds at the same rate of speed, and keeps the load at the same height, if so desired. This drum has a band brake by means of which the load can be sustained. The reversing lever, friction and brake levers, are all brought to a central position, so that the operator can work all of them in one position.

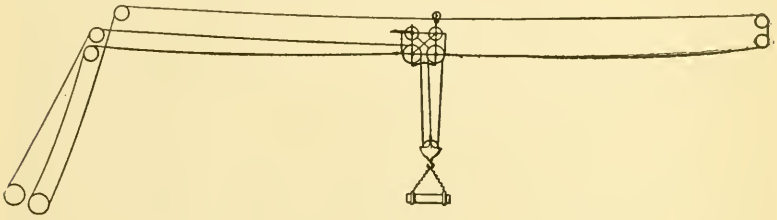


Fig. 9.

In Fig. 9 the method of leading the ropes on the long-span horizontal cableway is shown. The endless rope serves to hold the carriage in place while the hoist is being made. For horizontal motion the hoist rope is drawn in and the endless rope is payed out at the same rate of speed. As will be observed, the load may be hoisted or lowered at any point under the line of the cable, and the horizontal motion is given to the load at any height to which it may be raised.

The cableway at the Austin dam is thus described by E. W. Groves, Engineer in Charge of Construction.

"From the natural conformation of the ground this was an ideal place for the location of this piece of machinery. The west bluff rises nearly vertical to a height of 65 ft. above the crest of the dam, while the east bluff rises 10 ft. above the dam. This cable is suspended on two towers; the easterly one, in which is situated the hoisting engine, being 70 ft. high, and the westerly one 30 ft. high. The main cable on which the carriage moves is 2½ ins. in diameter, 1 850 ft. long, and 1 350 ft. between points of support. The load carried is about 6 tons.

"On the west side of the river and 1 200 ft. up stream from the dam is situated a limestone quarry, from which extended a gravity railroad along the west bluff directly under the cable. A car load of rock could be run from the quarry to the cable, taken up at once, and transported to derricks on any portion of the work. A double track was used near the dam, so that an empty car was ready to receive the empty skip, and a loaded car was also ready when the carriage came for a load.

"Under the cable at the easterly end of the dam extends the railroad bringing granite from the quarries, which can be taken from the cars

and transported to any portion of the work in the same manner as the limestone. Mortar is mixed on the wall near the workmen, sand and cement being taken out by the cable. Loads of about 5 tons are usually carried.

"This cableway is capable of supplying about 180 cu. yds. of material per day, where the distance to be traveled is not more than 1 000 ft."

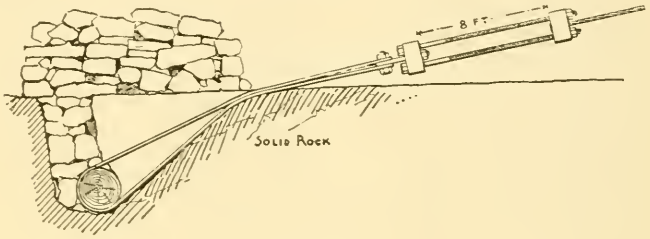


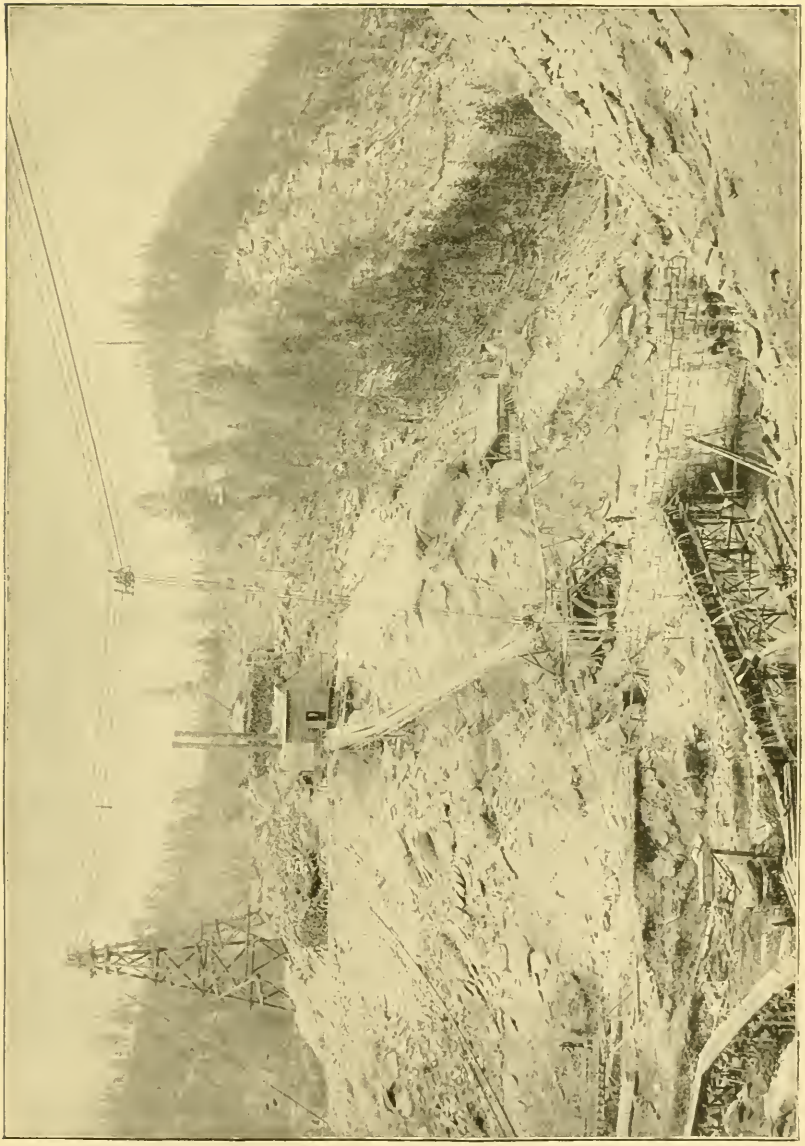
FIG. 10.

The heavy take-up device (Fig. 10) is not intended to be used in erecting the plant, but it is to take up the stretch occurring as the cable is used, or in raising the cable as the dams increase in height.

A great advantage of the cableway system is that when the plant is put in place it need not be moved or altered until the work is done. The frequent changes of position required where an ordinary contractor's railroad is used for moving material on such works increases largely the expense and time required. A criticism which might be made on the Austin plant is that many of the stones have to be handled twice, the cableway transferring them to derricks. This could have been avoided by the use of several paralleled cables, the load being taken out on whichever one was nearest to the place where it was to be deposited. A second advantage of the cableway is found in the fact that it is entirely clear of the work itself, and is safe from injury by flood or fire, and is reasonably safe from explosions from the work below.

The importance of this has been fully shown at Austin. No less than eight different times during a year has work on the dam been interrupted by high water, and at one time the water was 43½ ft. above low water-mark. Had the common plan of extending a railway out over the dam been adopted these delays would have been greatly lengthened by the time necessary to replace the portions of track and trestle-work washed away at these points.

Plate XLVIII shows the cableway used for building the Basin Creek



Dam for the water works of Butte, Mont. The dam is 120 ft. high, 300 ft. long, and was designed by Chester B. Davis, M. Am. Soc. C. E. The cableway spanned the dam and quarry 892 ft. between towers, one 80 ft. high, and the other 15 ft. high. Stones were taken direct from the quarry, and placed in the dam without further handling, and by the use of a snubbing post (Fig. 11) and horse all the stones were laid, no derricks being required or used.

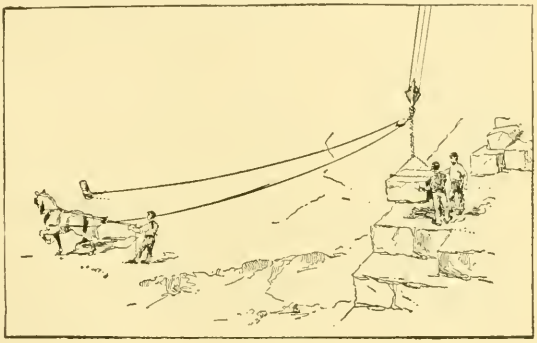


FIG. 11.

The cement mixer used on this work was moved by the cableway when it was necessary to put in a new foundation, requiring but a fraction of the time that would be required to do it by hand.

During sixteen days in June 1 430 cu. yds. of masonry were laid with an average of 86 men; this included six masons, quarrymen, firemen, and all labor employed about the dam and camp. This is an average of nearly 15 cu. yds. of finished work per day per mason.

The U. S. Corps of Engineers, under Lieutenant W. E. Craighill, used a cableway for building the lock and dam on the Coosa River, near Lincoln, Ala. The plant was erected in the spring of 1891; span, 1 000 ft.; main cable, 2¼ ins.; load, 8 tons.

The stone for this lock and dam was procured from a quarry about two miles back from the site of the dam; a 3-ft. gauge road was built and equipped with locomotives and cars. Each car held as its body a skip which was picked off the car by the cableway and by it delivered into place. The material used was brick and loose rock.

Plate XLIX shows a general view of the cableway at Point Pleasant, W. Va., taken when the water was very high. It shows to the right the coffer-dam surrounding the walls of the lock. The dam will lie

directly under the line of the cable. The distant tower is 100 ft. high. To the left is the head tower, located, as will be noted, near the quarry. The stone here is quarried and swung around underneath the line of the cable, and transported to the stone-yards shown in the center of the picture.

The contractors say they expect to use the cableway for work on the river 125 days each year, and work will last at least five years. The cableway will be used every day for transporting stone from the quarry to the stone-yard. The supplies and materials arriving by boat are unloaded upon the coffer-dam or upon the dock boat, and brought ashore by the cableway. It is found a great convenience in taking men across the river, as it is much more expeditious than ferrying them.

This plant has the distinction of being the largest hoisting cableway in existence, the clear span being 1 505.5 ft., main cable, 2½ ins. diameter; maximum net load handled, 4 tons. A great deal of material is also brought in over a railroad siding on the other bank of the river. A seam of coal underlies the quarry and an amount sufficient for all purposes is mined and transported directly by cableway to the dredges, hoisting engines and pumps in the river as well as to the boiler and electric-light plant.

At Rochester, N. Y., two cableways were introduced side by side, and about 60 ft. apart, for the erection of a bridge, span 630 ft. and towers 50 ft. high. A 30 H. P. 8¼ × 10-in. hoisting engine was used for each cableway. A system of electric signal bells was used for communicating with the engineer.

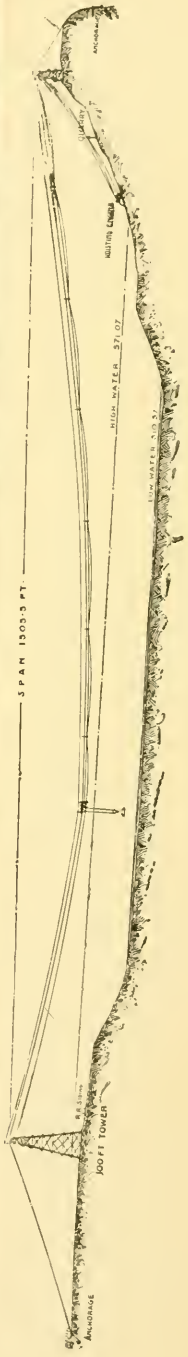
J. Y. McClintock, City Surveyor, furnished the following details of the operation and uses of this cableway.

The first stone was laid April 1st, and the last October 7th, 1893.

The estimated quantities for the bridge, all of which were handled by the cableways, were as follows :

Earth excavation	2 200 cu. yds.
Rock excavation	4 700 cu. yds.
Lumber in centers	350 000 ft. B. M.
Masonry in piers and arches	2 200 cu. yds.
Arch stones	2 278 cu. yds.
Concrete for spandril backing	2 660 cu. yds.
Coping	868 lin. ft.
Gravel	3 400 cu. yds.
Iron work	310 000 lbs.

PLATE XLIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 701.
MILLER ON CABLEWAYS.



In the iron bridge over the canal the floor beams are 40 ft. long, and weigh several tons. These were put into place by taking hold of each end by a carrier and working the two simultaneously. Stones were laid both in the piers and arches, not only under each cableway, but midway between the two; that is, 30 ft. distant at right angles from either. This was done by hitching the hoist from each cableway to the stone, and hoisting on one while paying out on the other, thus dropping the stone into exact position. This feature was extremely advantageous to the contractors in laying all of the heavy skewback stones on the last piers, when the high water had carried away the derrick frame which was used in laying the other piers, and as the water still kept high, it seemed unadvisable to build another frame for the few yards of stone remaining to be laid.

To lay the piers a frame was used which straddled the piers and on the top of which a traveler was employed. The cableways were used to bring the stone to the piers, and it was carried sideways by these travelers. After a pier was completed, this whole apparatus was taken hold of by both cableways and in 10 minutes was set over on the site of another pier. This was such a simple matter that in some cases where the stones intended for a given pier were not at hand the traveler was moved to another pier, and when the stone on hand was used it was moved back to finish the first pier. The wooden centers for the arches were made very heavy, and were framed on a horizontal platform and lifted into place. For this work also the cableways were found very handy.

On account of the limited space available, the cableways were used for pulling the old piers out and removing 5 000 cu. yds. of stone blasted from the bed of the river. They carried stone to the crusher and then delivered concrete to all points. They deposited the spandril filling, laid the spandril walls and coping, carried the curb and paving stones for the street service, and, finally, dragged out the immense mass of timbers used in the centers.

In Baltimore, L. B. McCabe & Company, contractors, are now building, with the assistance of a cableway, the stone arch, North Avenue Bridge, over the tracks of the Baltimore and Lehigh Railroad, and the Baltimore and Ohio Belt Line.

The span is about 800 feet, and the maximum load handled 4 tons (Plates L and LI). The view, Plate LI, is looking towards the head

tower, and shows an early stage of the work. Several lines of street cars run on North Avenue, and, while the work of construction is going on, traffic is carried on over the temporary bridge, shown at the left.

All the stone required in the piers is transported from cars near the head-tower over the tracks and cars, and deposited near the pier; derricks then place the stones.

The contractor had great difficulty in securing permission to place the tower in the street on account of obstructing traffic, and it is due to the same objection that two or even three cableways were not installed, so that derricks could be entirely dispensed with.

That the cableway is considered to be a safe device for transporting material is clearly evidenced by these views, showing the cableway spanning several lines of steam and electric roads upon which there is a large daily traffic.

The Baltimore Belt Railroad through the heart of Baltimore (Ryan & McDonald, contractors), was constructed in part by opening the street and building the side walls, one at a time. This was successfully done by cableways which were employed because they offered but little obstruction to street travel.

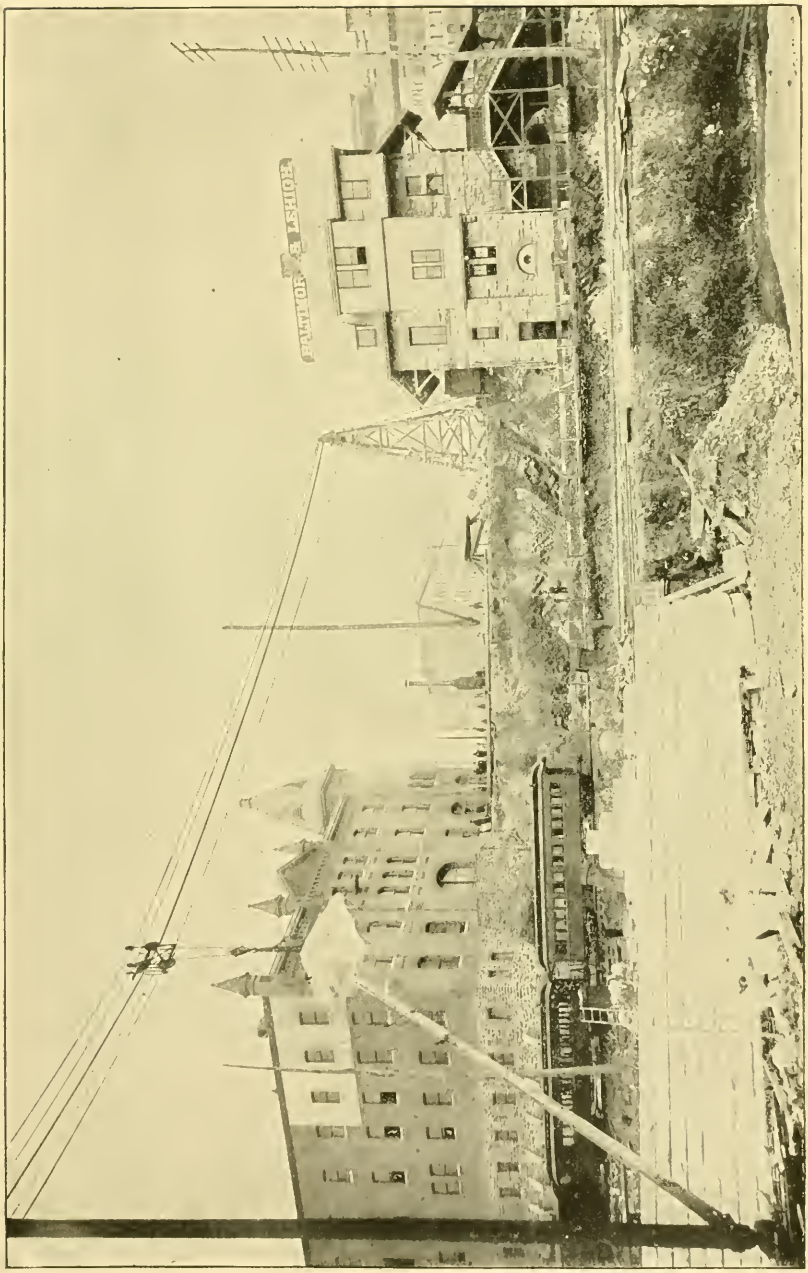
A description of this tunnel* says:

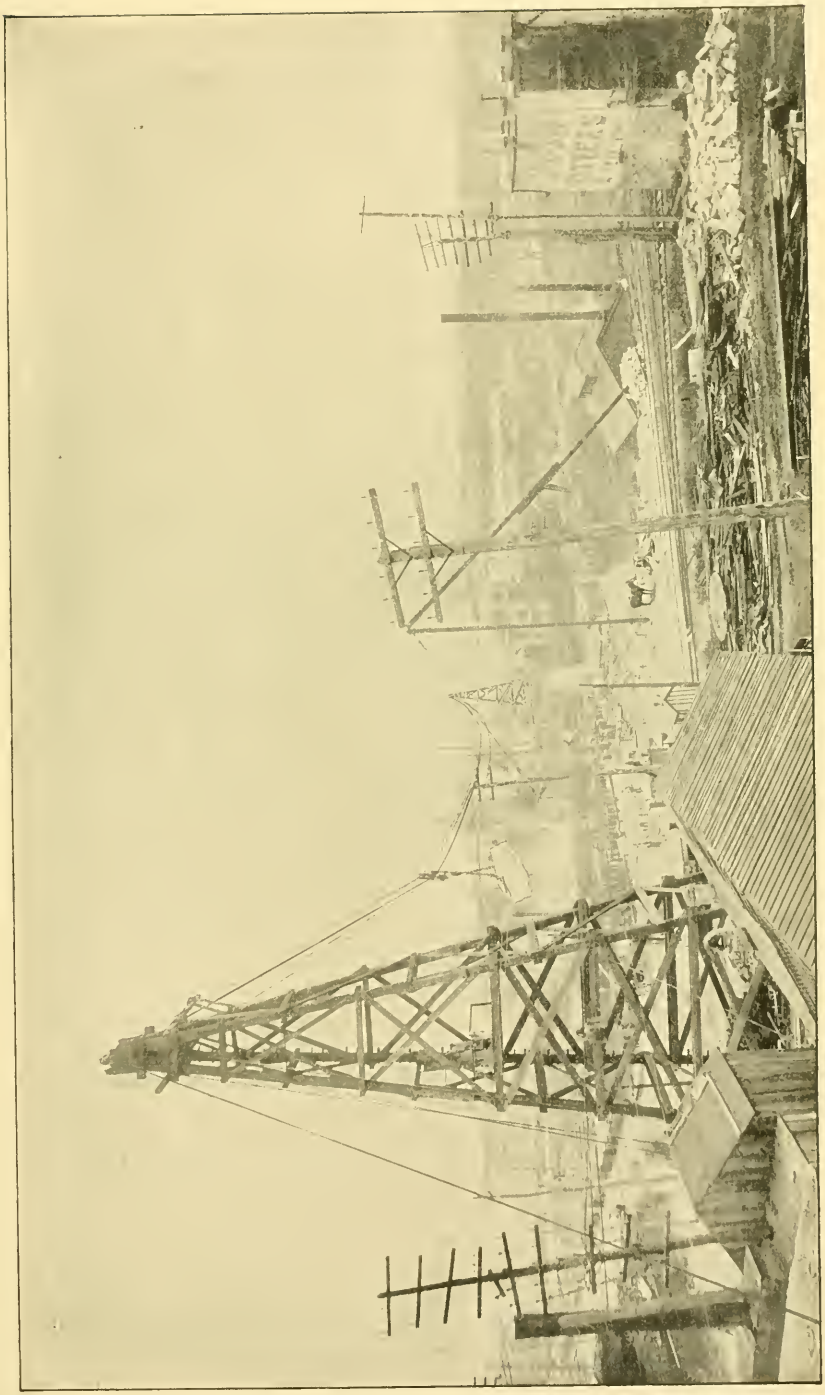
"An interesting feature of the work was the use of cableways in constructing about 1 200 ft. of the south end of the tunnel near Camden Station. This section was constructed by open cut in the manner to be described. As the street carries a very heavy horse-car and wagon traffic at this point it was impossible to open up the street the entire width necessary for the tunnel. Side trenches were therefore excavated and the side walls of the tunnel put in. These being finished the street over the tunnel was replaced by a timber structure, as fast as the material was excavated.

"It was in excavating the side trenches that cableways were used, placed directly over the trenches. They were used in covering both the excavated material and the stone, bricks, etc., for the masonry, it being found that stone could be set by the cableway without the use of the usual derrick. The advantages to be gained by the use of cableways in this case are obvious when it is considered that no hindrance is opposed to the street traffic, which can be carried on directly underneath the apparatus."

Recent improvements in buckets have developed a self-filling scoop bucket which, drawn up the slope of the bank, gravel, sand or coal,

* *Engineering News*, May 18th, 1893.


FABYAN'S MACHINE SHOP



will fill perfectly. The bucket is then conveyed along the cable and automatically dumped into a car or hopper. The buckets hold from one to two yards, and a load is taken every one and a half minutes, being conveyed 300 to 400 ft., and automatically dumped.

The cableway is quite extensively used in logging operations, and is known as the "skidder," or skidding cable-way. In its use a tree, located some 10 or 12 ft. from the railroad track, is allowed to remain unfelled. To this tree chains and blocks are attached, and it becomes the "head" of the cableway. The span is usually 750 ft., the main cable from 1 to $1\frac{1}{8}$ ins. Everything must be extremely light, because the moving must be done cheaply and quickly. This device does not pick up the load in the air entirely, but simply picks up one end of the log, or several logs, by tongs in the end of a single rope, and snakes them in with the other end of logs resting on the ground. They are simply raised high enough to clear obstruction.

The system calls for an auxiliary cableway, known as the "loading line." This has a block which is held practically stationary whereby by the use of an additional rope the log is picked up (after being delivered by the main cable) by the tongs, and delivered directly on the cars.

The time required after hooking on three logs up to the time they are delivered 700 ft. distant is less than a minute, and the sound of these logs rushing and crashing through the underbrush is enough to terrify an amateur.

A convenient form of cableway for logging in cypress is to use the mast of a scow as the head of the cableway. In this application of the system the logs are dragged in through the circle of 1 000 or 1 500 ft., and simply delivered into the water, where they are afterwards floated down to the saw mill.

The skidder is really the solution of logging in cypress swamps and where any other system would be impracticable. In its use for logging purposes, tramroads are placed in parallel lines about every 1 200 ft. apart through the woods. The skidder then works each side. The men operating it frequently stand up to the middle in the water and mud.

Traveling Cableways on the Drainage Canal near Chicago.—Portable or traveling cableways were invented some forty-three years ago by M. Pluchet, a Frenchman, and patent drawings show his portable

cableways used for building a canal. Whether it was ever put into use or not the writer is unable to state. The traveling cableway has developed many interesting things in its use.

Plate LII shows the traveling cableway, of which, at the present time, three are in use in the work at the Chicago Drainage Canal.

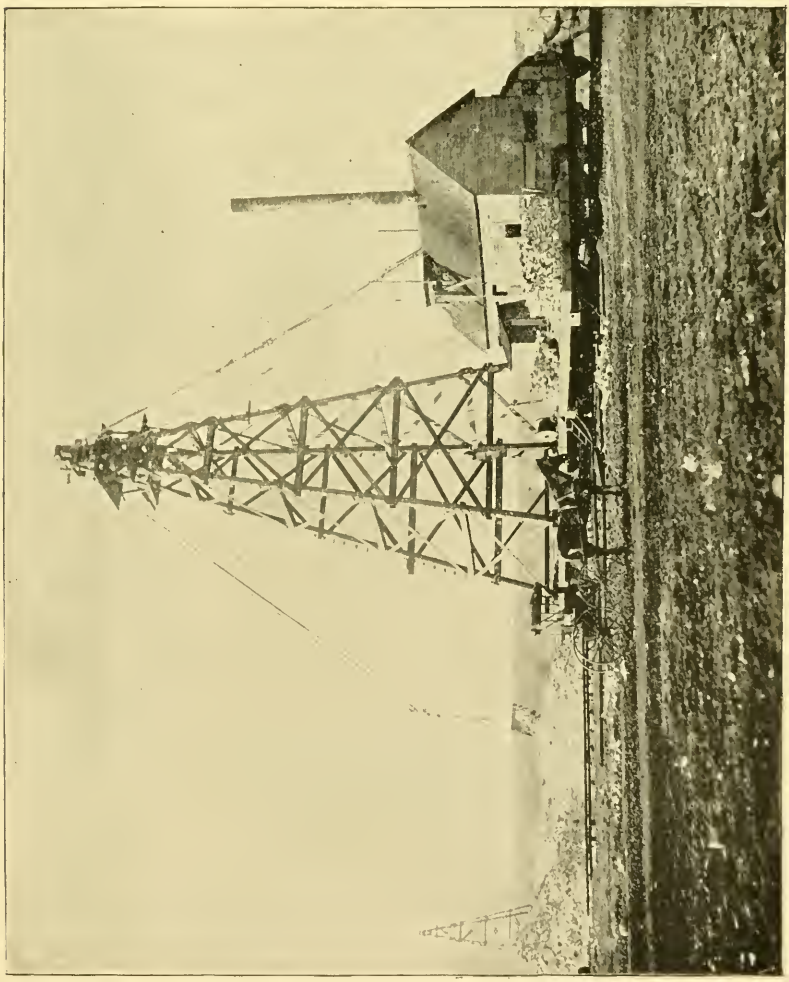
The span of this cableway is 623 ft.; the towers are 80 ft. high, and are located on cars 102 ft. long; each one of these cars rests on twenty-seven 33-in. wheels, and on this car or platform are located at the head tower the engine, boiler and the required amount of ballast, and on the tail tower simply sufficient amount of ballast for its stability.

The main cable is 2 ins.; the engine is double 10 x 12, 50 H. P., with double drums.

The cableway is used merely to take material from the canal and and deposit it in spoil bank on one side.

The work is usually done on a breast 12 ft. high. The record shows that 32 men at the breast, filling skips and chaining large stones, average 14 cu. yds. per man per day, or 28 tons of material. Of course, it would not exceed 11 yds. if it were not for the fact that a great saving of time is accomplished by the handling of large stones out of the pit by simply chaining them to the fall block. Stones weighing 6 or 7 tons have been handled out of the canal in two or three minutes, thereby making a distinct saving in cost for drilling, powder for blasting and the cost of sledging; and the mere act of throwing the chain around a heavy block is only a fraction of the cost of filling the skip. On the first 12 ft. of the canal the stone broke up fine under the blast; on the second and third lifts, it breaks up so large that the work seems to peculiarly fit the cableway, and a large saving is made on all material that is handled out in large blocks.

In handling large stones the danger of slipping a chain or breaking a link is minimized on account of the fact that the stone may be handled horizontally at any height desired, just sufficient to clear the surface of the ground if required. The usual practice is to deliver these heavy blocks on the canal slope of the spoil bank, and they are frequently taken out at the noon hour with only two or three men at work, on extra time, so that when the men come in to fill the skips they can find nothing but uniformly broken material to handle. Many contracts were taken along the canal for channeling, blasting, hoisting,



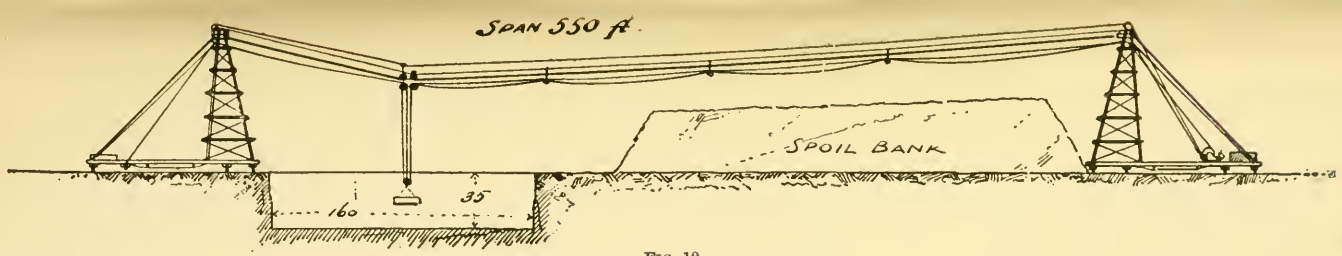


FIG. 12.

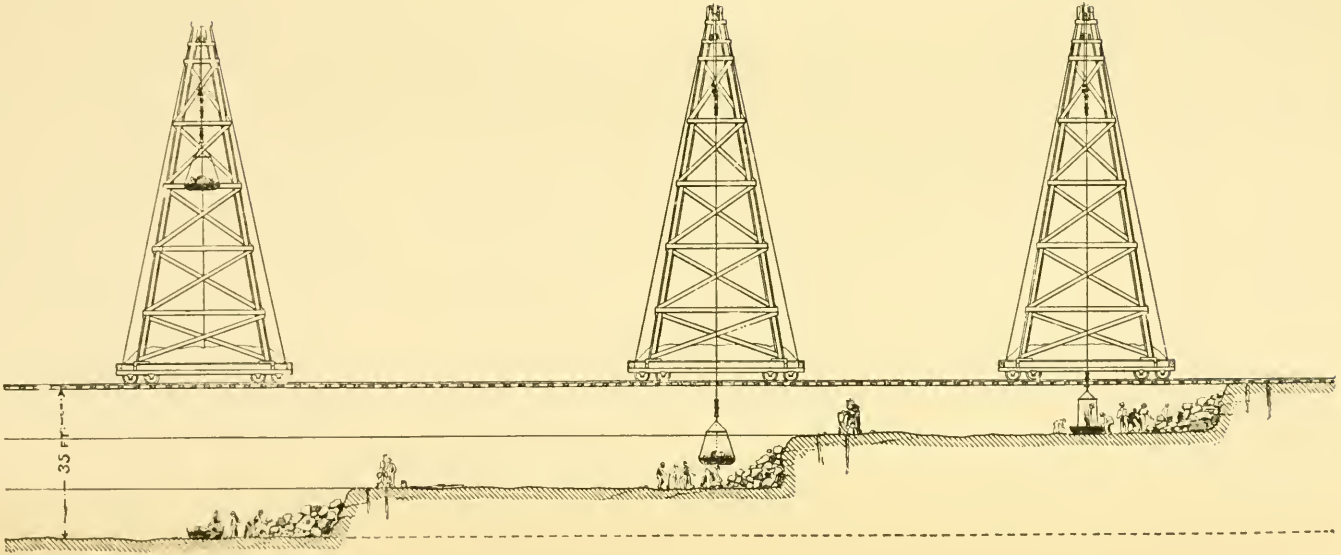


FIG. 13.

conveying and delivering the material to the spoil bank for 80 cents per cubic yard, working down to a depth of 35 ft., and 160 ft. wide. The facts are that probably not all of the contractors at this price are making money, but, nevertheless, it is certainly not to be disputed that on the sections that are properly worked, with the proper organization and proper machinery, so that the machines are constantly used to their maximum capacity, there is money to be made at 80 cents per yard, and such contractors working at this price are satisfied to take more work at the same price.

The aerial dump (Plate LII) has recently been developed which has increased the capacity of the cableway enormously. The idea is not new, but the peculiar application, however, is novel. The auxiliary rope arranged to draw up the bucket or skip is installed, passing from the carriage to the engine, parallel to the hoist rope, and winds on the same drum as the hoist rope. Therefore, when it is desired to dump the load it is merely required to draw in this rope at a little higher speed than the hoist rope, thus the material delivers without delay to the cableway. The instant the load has been spilled from the skip the carriage returns to the canal. For obvious reasons, any authentic records are difficult to obtain, but the following are understood to be correct. The record for the month of November was 230 skips daily. The greatest day's run was 258 skip loads of broken stone, averaging $2\frac{1}{8}$ yds. each, and 12 large stones averaging 2 yds. making a total capacity of 572 yds., or 1 144 tons. A few days ago the writer received a report that this cableway handled 302 skips in 10 hours, or 600 yds. per day; 500 yds. seems, however, to be about the average.

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TRANSACTIONS.

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702.

(Vol. XXXI.—April, 1894.)

AN INSTRUCTIVE EYE-BAR TEST.

By A. C. CUNNINGHAM, Assoc. M. Am. Soc. C. E.

READ MARCH 21ST, 1894.

WITH DISCUSSION.

From a lot of nine steel eye-bars one was to be selected for test.

The bars were apparently perfect with the exception of one, and in this bar one of the finished heads contained three seams. It was decided to take this bar for testing, as the seams condemned it for use, and in the event of failure on account of the seams, it could be reheaded and retested.

The nominal size of the bar was $6 \times 1\frac{9}{16}$ ins.

An elastic limit of 35 400 lbs. per square inch was developed, and the bar broke in the head, as indicated at F in Fig. 1, at a stress of 41 500 lbs. per square inch.

The fracture was square and granular, and started in a seam, as indicated at S.

The body of the bar gave no elongation or reduction.

The excess of area on the line AB over the body of the bar was 50 per cent.

The broken head was cut from the bar and enough of the body to reach sound material, after which the bar was reheaded and reannealed. An examination showed that the new head contained seams, quite a decided one being at the point *S*, Fig. 2, which was chipped out and found to be about $\frac{1}{8}$ in. deep.

At a stress of 60 200 lbs. per square inch a piece broke out of the new head, as indicated in Fig. 2.

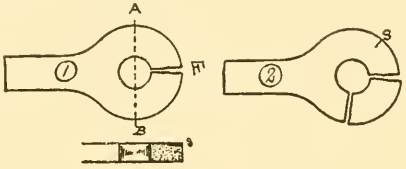


Fig. 1. Fig. 2.

The fracture was square and granular, and in solid material, without visible flaws or seams.

On the second test, the bar gave an elongation of 7% in the body.

The specimen test representing this bar had given results as follows: Elastic limit = 36 600; ultimate strength = 63 100; elongation in 8 ins. = 30%; reduction of area = 49 per cent.

The object of full-sized eye-bar tests is to check the quality of the workmanship, the condition, quality and grade of the metal.

In this case the specimen test representing the bar indicates a a metal of the proper grade and of excellent quality. Was the workmanship and treatment of the bar defective, or is the specimen test misleading?

Even if the workmanship and treatment were defective on the first head, when the bar was reheaded especially for testing, it may be assumed that the best of care would be taken, and yet the failure was worse than in the first test.

What, then, is wrong? The following analysis of drillings taken from the face of the second fracture, Fig. 2, shows the cause of the failure: Carbon = .21%; phosphorus = .118%; manganese = .60%; sulphur = .126 per cent.

The phosphorus is high, nearly .02% higher than the limit which is set for structural steel, but probably not higher than in at least 30% of the material which is being made and used at the present time.

The primary cause of the failure is found in the sulphur, which is

over twice as great as the recognized limit for material which is not to be reheated, and over three times as great as the limit for forgings, with which eye-bars will class.

The probabilities are that this eye-bar was from the extreme top of an ingot where the impurities due to segregation are in excess.

With the knowledge of this high sulphur in mind, we can surmise what the trouble was with this bar. The sulphur at the end was probably just at the critical point beyond which the steel would have crumbled in rolling. In making the head on the bar, a small additional amount of sulphur may have been absorbed in the heating, and the severe nature of the work in forming the head was sufficient to severely injure the steel.

The behavior of this bar under test is a strong plea for the cropping of ingots. If the upper quarter or third of the ingot was discarded, the highly segregated portion would be removed, and the remaining portion would be fairly uniform, or at all events not dangerously segregated, as the top of an ingot is most likely to be.

The cropping of ingots has always been advocated for the removing of piping and unsoundness. With the knowledge of segregation which has been generally diffused during the past few years, the additional need of cropping seems almost imperative.

DISCUSSION.

Percival Roberts, Jr., M. Am. Soc. C. E. (by letter).—Before being able to draw a final conclusion, we should have more definite information as to the following points:

Was the specimen tested cut from the same bar from which the full-sized eye-bar which was tested was made?

What was the analysis of the specimen tested?

Were analyses made from different parts of the full-sized bar to determine the matter of whether the analysis of the broken head gave only local results, or were these results obtained also at other points in the bar?

Were any analyses or tests made from other bars made from the same melt as the bar in question?

Was the steel Open Hearth or Bessemer?

What were the conditions existing during the annealing of the bar? Also, what process was used, the character of the furnace, and the

nature of the fuel. Also, length of time the annealing took place, and how long afterwards before the bar was removed from the annealing furnace ?

The character of this steel was such that it would be an extremely sensitive material to the effects of annealing, which should have been very thorough to produce beneficial results.

Mr. Cunningham's suggestions as to the cropping of ingots would cause such an increase in the cost of material as I fear not many customers would be willing to pay. Hitherto, the Government alone, for ordnance purposes, has enjoyed such a luxury as this. I do not believe it is necessary for structural work, as, by the use of a proper quality of material and by proper manipulation, no such segregation should take place as is noted in the present instance; although, I much question whether the instance here referred to is segregation, unless we have additional information or answers to the questions I have above asked.

By keeping the metalloids in the steel sufficiently low there is little practical danger of segregation which amounts to anything taking place. The extended use of Bessemer steel in this country has caused the use of material entirely too high in phosphorus for eye-bar purposes. I believe in no instance the phosphorus should exceed .08, but better .06, while sulphur should not be over .06. With these metalloids at these points and carbon about .16 or under there would be very little danger of segregation.

James Christie, M. Am. Soc. C. E. (by letter).—The analysis of the steel in the eye-bar head showed phosphorus and sulphur higher than is considered acceptable, but it is not uncommon to get satisfactory tensile tests from steel of similar composition. If these elements were found to be less in the opposite or unbroken head it would be safer to assert that the fault was due to segregation, and that the analysis did not represent the general character of the steel.

Porosity of the ingots, or cracks developed by blooming, might account for the seams in the defective head.

It will be observed that at the second test, the body of the bar must have been strained to the verge of rupture; the small difference in unit tension between the full bar and the test piece is a frequent result, possibly due to annealing.

I do not question the advantage of liberal top cropping of ingots to remove unsound or segregated material. Upset eye-bars are liable to fail in the head if not properly annealed. The steel should be uniformly heated to a temperature of about 650° Cent., and uniformly cooled.

R. H. Thurston, M. Am. Soc. C. E.—I imagine that the writer of the very pithy paper before us has stated the real cause of the defective quality of the eye-bar examined and tested. The cause of failure

and that of the seams described are probably identical. The phosphorus is certainly too high, and the total of hardening elements altogether too great; but the peculiarity of the analysis is in its large percentage of sulphur. Not only does sulphur cause "hot-shortness" by its volatility and tendency to diffuse itself at a welding heat, and by producing, when hot, a resultant stress throughout the mass of material affected by it; but it also gives decided tendency, I imagine, to the production of crystals throughout the whole of the metal thus affected. Very possibly this may account both for the brittleness and the crystalline structure of the steel; for it is well known that overheating is very apt, especially in metal containing sulphur, to produce just such crystals. In this case we have excess of sulphur, as proven by analysis, and overworking, with probably overheating, or at least too much work and too much heat for this particular metal, and their natural results in brittle steel and overstrained and defective heads.

I have an idea that such metal as this could be much more safely worked by the hydraulic forging machine—where it must be used at all—than by any system of hammering or rolling. I found the press system of forging in use, many years ago, in Vienna, where Mr. Haswell, the English manager of the State Railways, was employing it in making all sorts of running parts of machinery and rolling stock, and I was so much impressed with it that I introduced some account of it into my report to the Government on the machinery and manufactures of the time, and especially of the Vienna Exposition of 1873. But a much more complete account of what was then already accomplished is given in Blake's report, in the same volume, on Iron and Steel. The slow and steady flow produced by the press has a vastly different effect upon the metal than the quick, and often comparatively concentrated and superficial, action of a hammer, particularly if the hammer is too light for its work; and the encouragement by engineers of the substitution of the press for the hammer will, I am inclined to think, render available for structural and other work some qualities of metal that, to-day, must be condemned. I doubt if there is any serious objection to sulphur in quantities that are not too great to permit working the steel. In fact, it is said that, in the manufacture of ordnance, the Swedes have introduced small doses of that element to give toughness. I doubt if its presence, however, is to be considered an advantage, ordinarily; but it would be an advantage, could ores and irons and steels containing moderate doses of that substance be brought into use safely.

The avoidance, meantime, of such materials; the careful cropping of the ingot regardless of the magnitude of the wasted part; and the adoption of better methods of working are continually making us safer against the accidental introduction of such metal into structures

As illustrating the fact that simple heating, without heavy strain or

loading, will sometimes produce this crystallization or granulation, a sample bit of wrought-iron bar in my collection has interest. It is a piece of 2-in. bar, which once formed part of the grate under a gas retort. It was always hot, never what would ordinarily be called overheated, and was turned a half or quarter of a turn whenever, after considerable periods of time, it sagged too far on of line, as grate bars always will sooner or later. After some months of use, it was taken out and broken, and found to be a mass of large crystals throughout the whole length subject to heating. To-day after, perhaps, three years' exposure to the air, these crystals remain untarnished; showing either the completeness with which their affinities are satisfied, or the

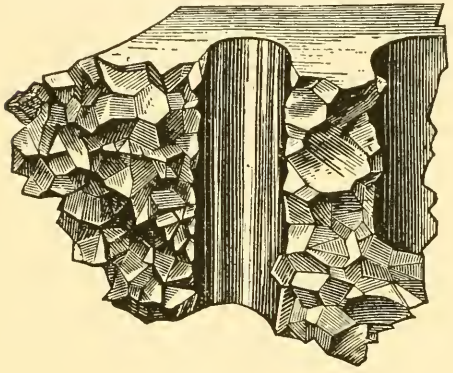


FIG. 1.

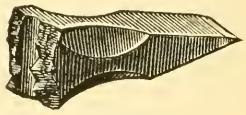


FIG. 2.

fact that a crystalline structure gives immunity from oxidation through some other action. At page 581, of Vol. II, of my "Materials of Engineering," is a remarkable illustration of the same sort—a hammer-head of steel, after exposure by accident to prolonged but moderate heat, became similarly a mass of crystals. The cut above is made from a photograph, and is a faithful reproduction. Fig. 1 shows hammer-head magnified; Fig. 2, hammer-head natural size. The effect of heating and forging, as presumed, are very strikingly illustrated by other engravings in that article. The repeated blows of a hammer undoubtedly facilitate this formation of crystalline structure. It seems more than possible that the action of the forging press may prove very much less objectionable on this score; though, after all, at present, it may be perhaps as well that anything that cannot be safely hammered should be rejected.

CHARLES MACDONALD, Vice-President, Am. Soc. C. E. (by letter). —The paper recalls the familiar subject of the relative importance of chemical and physical tests, of which so much has been said and written that it is scarcely necessary to indulge in extended discussion.

Even assuming that the hypothesis as to the chemical constituents of

the bar in question is true, what inference may be drawn from it? Shall the engineer fix limits of carbon, phosphorus, manganese, etc., for the manufacturer, and thereby relieve him of responsibility for the physical properties required, or shall he simply indicate what the quality of the finished product must be, and leave to the manufacturer to determine the best methods of accomplishing the result?

There can be but one opinion as to this. The engineer has to deal with results and not methods of manufacture, and the only satisfactory method of verifying results is by physical tests.

The writer observes that the object of full-sized eye-bar tests is to check the quality of the workmanship, the condition, quality, and grade of the metal.

In so far as the workmanship refers to forging the eyes, this is correct; but the condition, quality, and grade of the metal should certainly have been determined before any expense was incurred in the bridge shop.

This would seem to be a self-evident fact, and yet, judging from some of the best bridge specifications, it is not properly appreciated.

In this connection reference might be made to an extract from the "Hawkesbury" specifications, from which it will be seen that, while the chemical question is ignored, full provision has been made for determining the quality of the material beyond peradventure.

These specifications represent the best practice in England at the time they were drawn up (1886). For brevity and simplicity they are worthy of careful study:

HAWKESBURY SPECIFICATIONS. STEEL WORK. 1886.

"37. The whole of the superstructure will be "Siemens" steel, of a make specially approved by the engineer.

Strips cut lengthwise or crosswise to have an ultimate tensile strength of not less than 30 tons (2 240 lbs.), and not exceeding 33 tons (2 240 lbs.) per square inch of section, with an elongation of at least 20% in a length of 8 ins. The angle and bar steel to stand such forge tests, both hot and cold, as may be sufficient, in the opinion of the engineer, to prove soundness of material and fitness for the service. Strips cut lengthwise or crosswise, 1½ ins. wide, heated uniformly to a low cherry red, and cooled in water of 82° Fahr., must stand bending double in a press to a curve of which the inner radius is one and a half times the thickness of the steel tested. The ductility of every plate, angle, etc., is to be ascertained by the application of one or both of these tests to the shearings, or by bending them cold by the hammer. All steel to be free from lamination and injurious surface defects. One plate or angle, etc., to be taken for testing from every invoice, provided the number of plates, angles, etc., does not exceed fifty. If above that number, one for every additional fifty or portion of fifty.

Steel may be received or rejected without a trial of every thickness on the invoice. In cases where plates or bars have to be heated, the greatest care must be taken to prevent any work being done upon the material after it has fallen to the dangerous limit of temperature

known as "blue heat," say from 600 to 400° Fahr. Should this limit be reached during working, the plates or bars must be reheated.

38. All sheared edges of plates and bars will have at least $\frac{1}{16}$ in. of metal removed by planing, and all holes will be drilled; but a hole $\frac{1}{8}$ in. smaller than the finished hole may be previously punched, if desired.

42. Tests of full-sized bars will be made as follows: Out of every lot of 103 bars, three shall be selected by the inspector; if two out of the three break in the body of bar with a stress not less than 28 tons per square inch, nor a less elongation than 10% in the length of the bar, then the lot of 100 will be accepted, if otherwise satisfactory in quality. If but one out of three so breaks, the testing shall be continued at the expense of the manufacturers until the required proportion of $\frac{2}{3}$ be reached. But if five bars break consecutively in the head without developing the above-specified qualities, then the whole lot of 100 shall be rejected."

It may be added that in the present state of the art it is the exception to find steel eye-bars breaking in the head. The requirements in this regard could, therefore, be materially advanced.

L. L. Buck, M. Am. Soc. C. E.—In all the tests of steel and iron which I have made, I have felt that I was unqualified to judge of the effect of chemical constituents because of not being a metallurgist, but when I had specimens rolled to the various thicknesses and shapes to be used in my work, and having cut test specimens from them, had tested them for elastic limit, stretch, reduction, bending, upsetting and forging, and texture of ruptured section, and have assured myself that only such blows as met all the requirements are used in the work, I have found the result generally satisfactory, and assumed that the chemical constitution must be good. I do not mean to say that I would discourage chemical analysis, but that I should be obliged to leave that to some other person. It appears to me that the proper party to ascertain the best chemical combination is the manufacturer. At any rate, he must attend to it, or he will fail to give satisfactory materials.

I have said in other discussions that with all the tests of specimens to ascertain the physical properties, of steel especially, the engineer should insist upon the inspection continuing through the work in the shop, watching all operations of punching, shearing, boring and planing. Also the forging, upsetting, etc., while the metal is hot, and the testing of occasional full-sized members. His specifications should require the rejection of such pieces as do not behave satisfactorily in undergoing such treatment. This calls for close attention on the part of the inspector, but it will pay for the trouble. It will also cause the manufacturer to look carefully after his chemical combinations.

J. M. Knap, M. Am. Soc. C. E.—One is not at all astonished at the results of these analyses. With the large amount of phosphorus and the large amount of sulphur it seems to me a very natural thing that this should give way, especially if it was treated to a high heat; I think high heat would cause the cracking of the eye-bar.

To go back about 25 years, I might state, to show that the physical properties of iron or steel are more important sometimes than the chemical, an incident that came under my observation. In the case of the St. Louis Water Works, in which I was engaged in building the engines, the specifications called for a certain tensile strength of the cast-iron walking beams. The tensile strength required was, I think, 30 000 lbs. They had heard that the Rodman guns required some such test, and thought for these large beams, 30 ft. long and weighing about 30 tons, if they were of a tenacity of 30 000 lbs., it would be a good thing; it would have been under some circumstances. We tried to change this condition, but the Commission insisted upon it, requiring us to make the beams of iron that would stand this test. We were obliged to make the iron of very high density in order to secure the tensile strength required, but always found there was a great deal of shrinkage under such circumstances, and informed the engineers, and the commissioners especially, that there would be danger in making these beams, but were obliged to do it. We cast one of the beams, and in course of time uncovered it carefully, cooled it slowly, and finally took it to the planing machine to plane off the hubs, when one of the men, who was a very strong, muscular fellow, in chipping off the sinking head, hit it such a hard blow that this walking beam went in two pieces, flew apart and separated about three inches. The force that was required to break that beam, calling for 30 000 lbs. to the square inch, was a good many hundred tons. The thing was tried again and a second beam cast. That gave us less trouble because it broke in two in the middle as we were lifting it out. The web cooled first (causing the periphery or rim to burst with the great shrinkage strain) just as a large fly wheel would break if cooled too rapidly. We tested specimens out of these two beams, and the commissioners were satisfied. We afterwards made the beams according to our own ideas, out of softer iron with a tensile strength of 22 000 lbs., and they are working to-day.

I relate these facts simply to show that practical experience should go hand in hand with theory. The specifications requiring the walking beams to be made of cast iron, with tensile strength of about 30 000 lbs. per square inch, would have been all right for a heavy housing shaft or crank, or for heavy ordnance (of that day) when the proper cooling and annealing processes were provided, So it is, in my opinion, with steel and wrought iron for structural work. Certain percentages of phosphorus and sulphur which may give entirely satisfactory results in some forms may cause failures in others.

The contractor or manufacturer, especially if he assumes the responsibility, should be allowed to use his experience and his judgment in such cases.

Wm. De H. Washington, Assoc. M. Am. Soc. C. E.—It seems to

me there is a medium to be struck between the two sides of this argument; that is, whether we shall have too much chemistry in our specifications or none at all.

What one of the gentlemen just said, may well be taken into consideration. He tells us of a large casting in which the iron was of the requisite tensile strength, but so brittle that it broke of its own weight in one case, and by a slight chiseling with a small hammer before it was put in place in another. Had this been put into position a serious accident is likely to have been the result, as it surely would not have withstood the slightest shock. While this was cast iron and the material we are treating of is steel, nevertheless, it seems to me, the same principle holds good, that some requirement from a chemical standpoint should be made in addition to a tensile strength requirement, to guard against the effect of the natural vibration in a bridge, which must surely have a serious effect, and in the course of a few years on hard or brittle iron or steel, be very likely to render it unsafe, by crystallization, to life and traffic, which it is the engineer's business to guard. It therefore seems to me that some reasonable requirement should be made in regard to the chemistry of all iron and steel for use in bridge work in drawing our specifications for such structures.

G. H. Blakeley, Jun. Am. Soc. C. E.—I think, if the analysis represents anything, that the character of the material was very inferior. I know in my own practice, even where there is no specified limit in chemical composition, I do not permit any steel to be used that is higher in phosphorus than .08. We have proved that by experience, and the law is not to use any steel that is greater than .08 phosphorus. I think that, so far as the specification by engineers of the chemical composition of steel is concerned, they are apt to carry it too far. If they would limit themselves simply to the specification that for acid open-hearth steel phosphorus should not exceed .08 or .07, and for basic .04, I think that would accomplish everything that could be desired. In the furnace I believe that the chemical action of the sulphur and phosphorus go on about jointly, and that if the phosphorus is reduced to .08 the sulphur will be about .05 or .06. We seldom pay any attention to the sulphur, because it always keeps below .06. I believe that every railroad company building bridges should specify a steel that does not exceed .08, and I believe it is their prerogative to use their best diligence to see that the specifications are executed by competent, continuous inspection of every heat of steel.

I have had some experience with large eye-bars in which the chemical composition was entirely satisfactory, and yet they failed just about as badly as these did, but I believe that was due to the size and lack of proper power in the rolling machinery to compress the section in its manufacture. But I believe, if .08 is set as the maximum limit

for phosphorus, that you will seldom have the unfortunate result that was produced at the testing of this bar.

Lest I be misapprehended, I wish to say that I do not mean that the engineer should specify a complete chemical analysis. If he specifies the phosphorus and sulphur, for instance, he must not touch the carbon and manganese, but must leave that to the manufacturer; if he contents himself with the phosphorus, I think he covers the case.

George S. Morison, M. Am. Soc. C. E. (by letter).—It seems to me that on many accounts this test is less instructive than is claimed. The bar broke in consequence of visible flaws, as was to be expected. After being strained beyond the elastic limit the fractured head was cut off and a new head forged on the stretched bar. The bar was then reannealed, retested, and again broke in the head, showing flaws. A head forged on a bar which has already been strained beyond the elastic limit cannot be considered a fair test, as before forging such head the metal has been put into an abnormal condition. I have had many bars reheaded and retested, but have found these tests more instructive as to forging than anything else. I have found they could not be depended on as showing the quality of the material. In this case, although the bar broke in the head, it did not break at the most prominent seam. Had it been entirely sound the chances are that it would have broken in the head.

When bars are defective requests are always made by the manufacturer to have them taken for test bars, with the understanding that the test bar is acceptable if it meets the specifications, and that if it fails through some of the visible defects, it shall be thrown out and another bar substituted for it. This is really very objectionable, as it establishes a practice of using bars for tests which do not represent the actual material used. To avoid this I have inserted the following clause in my recent specifications :

"No bars known to be defective in any way shall be taken for test bars, but the bars shall be selected as fair average specimens of the good bars which would be accepted for the work."

Bars with visible surface defects are unfortunate accidents which none of us want; in some cases they are good, but if of steel and the defects are in the head, they are almost invariably bad. One trouble that I have found in steel manufacture is the attempt of makers to make something which, while actually reaching the literal requirements of the specifications, is a cheaper material than the specifications intended to provide for. To prevent this, I always put into my specifications requirements about phosphorus and some other matters, solely to avoid an attempt at making a material which has large chances of failure. The same thing may be said of bars known to be defective. These tests may be instructive, though less so than is commonly supposed, but they amount usually to making tests which have to be duplicated, and to losing time.

William Metcalf, Past Prest. Am. Soc. C. E. (by letter).—It is difficult to discuss this paper from a steel-maker's standpoint, because of lack of information on just the points that an engineer cannot give.

The analysis is bad enough, but if the steel was not too red-short to work sound, the sulphur would not cause cold-shortness if general experience goes for anything.

The quantity of phosphorus and of manganese would account for any cold-shortness there might be in a steel of higher carbon.

There can be no question that segregation does occur, but I do not think it is necessarily at the upper end of an ingot. I believe it will be most pronounced in that part of the ingot that sets last, and that may be the middle part, or even lower down in some cases.

The absence of any knowledge of the appearance of the fracture makes it impossible to say whether the head was forged properly or not; there may have been a very uneven heat in the steel which would have been sufficient to cause the break.

Again, we do not know how the steel was made; if it was blown clear down, or melted clear down, and then recarbonized back to .21 carbon by the use of Fe Mn, it may have been left rotten and red-short by an excess of oxygen, or nitrogen, or both, which the manganese did not remove.

I believe that few engineers, and not all steel-makers, realize the irreparable harm that is often caused in this way; the steel is full of gases, the ingots are spongy, and the steel is often too red-short to be workable.

By melting, not lower than .15 carbon, or .12 at the least—I prefer .15—and then using just enough Fe Mn to remove as much oxygen as is possible and to give .20 to .25 carbon, a vastly better product will be had than if the same material had been melted clear down and then carbonized to the same point.

In open-hearth practice there is a distinct advantage to the manufacturer in stopping the melt at .15 carbon; it takes less time, less fuel, less wear and tear of furnace.

There is less gas, less danger of boiling, sounder ingots, and, consequently, less scrap.

Whether it would be possible for an expert in Bessemer practice to stop the blow at about .15 carbon, I am not competent to say, but if he could do it, he would gain just as much as can be gained in the open hearth.

I think Mr. Cunningham is right in asking for good chemistry and sound steel; unfortunately, chemistry does not tell us about the oxygen and nitrogen which we know to be harmful when in excess.

In regard to cropping the ingots, as Mr. Cunningham suggests, it would be a serious expense to cut away one-third of every ingot, and an entirely unnecessary one if the ingots were made with any skill.

In good practice, 10% is an abundance to allow for pipe, and beyond the pipe there can be no good reason for cutting away the steel.

It is apparent that in the case under discussion the pipe was all cut out, or it would have shown in the fracture, and then there would have been no need of a discussion.

I will give now two ideal analyses and a physical test of the steel:

Silicon	.007	.009
Phosphorus	.005	.010
Sulphur	.024	.027
Manganese	.060	.050
Carbon	.12	.090

Elastic limit, pounds per square inch	30 990
Tensile strength, pounds per square inch	46 800
Elongation in 2 ins., 82 per cent	41
Reduction of area, per cent	75.85
Fracture silky, ½ cup.	

Here you have a comparatively high elastic limit, low ultimate, and a delightfully high ductility.

The steel was made to order to replace iron, and it answered its purpose.

It works beautifully at a high iron welding heat, and it is so red-short at and below medium orange color that it is a nuisance to handle it.

Surely the sulphur is not at fault here.

To make the steel at all it had to be melted down to nearly zero, and then be recarbonized just enough to make it so that it could be worked.

I have the test pieces before me; there is not the slightest evidence of a flaw in the fracture.

I should dislike to have this steel in a boiler or any structure of mine.

In comparison to this I call attention to 24 tests of boiler sheets, with the mean analysis.

TWENTY-FOUR TESTS OF BOILER SHEETS, APRIL, 1884.

	Tensile strength. Pounds per square inch.	Elongation. Per cent. in 1 in.	Reduction of area. Per cent.
Highest	85 680	27.00	50.5
Lowest	64 640	42.00	62.3
Mean of 24	73 142	33.96	48.0

Quenched from bright yellow heat, only one piece endured the cold bending test.

Mean analysis drillings taken from every sheet and mixed thoroughly:

Silicon	.014	Manganese	.204
Phosphorus	.050	Carbon	{.425 / .446} .435
Sulphur	.023		

Boilers, 42 ins. diameter; 26 ft. long; two 16-in. flues; shell, ¼ in. thick.

These boilers have had ten years of steady, hard work, being pushed often to their highest capacity. They are in perfect condition and look as if they would never wear out.

Such steel would probably not do well in a locomotive fire-box.

It is crucible steel, made by a concern having a high reputation for that sort of work; not ours. Although this steel seems high for boiler work, compared to many specifications that are sent out. I feel much safer with them than I would with boilers made of the milder steel with which they are compared.

The ductility seems good in this boiler steel, but it was measured on 1 in., and the testing engineer notes: "These figures are all probably too high, owing to the shape of the test piece required by the Government."

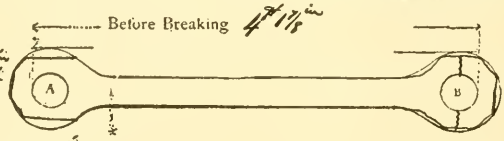
A comparison of the mild steel given above, with some high steel, may be of interest. The three tests, pages 428 and 429 (on Keystone Bridge Co. blanks) in eye-bar form were made at the request of some friends.

TEST No. 1.

FULL SIZED TEST OF 2 × 1 in. Steel EYE BAR.

Rolled by _____ Manuf'd by Crescent Steel Co.

Before Breaking 4ft 1 7/8 in.



Diam. of Pin Hole 2 in.	Diam. of Pin Hole 2 in.
Elong. of Pin Hole ____	Elong. of Pin Hole ____
Nominal Section 2 in. × 1 in.	Actual Section 1.91 × 1 in.
Original Area 1.91 sq. in.	Fractured Area ____
Gauge Reading for Elastic Limit 6.25 = 162,500	Pounds.
Gauge Reading for Ultimate Strength 7.40 = 192,400.	Pounds.
Elong. in 42 in. Inches 1/4 in.	Elong. in ____ Feet.

ELASTIC LIMIT	85,078	POUNDS PER SQUARE INCH.
ULTIMATE STRENGTH	100,733	POUNDS PER SQUARE INCH.
ELONGATION IN 42 in. INCHES.	0.5	PER CENT.
ELONGATION IN ____ FEET		PER CENT.
REDUCTION OF AREA AT FRACTURE		PER CENT.

Fracture Broke in head. Close grain

Analysis

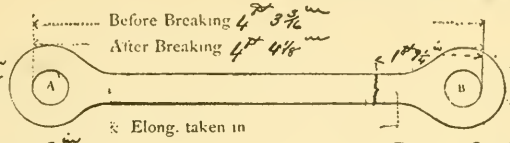
Silicon	.156
Phosphorous	.008
Sulphur	.015
Manganese	.24
Carbon	1.35

REMARKS

TEST No. 2.

FULL SIZED TEST OF 2 × 1 Steel EYE BAR.

Rolled by _____ Manuf'd by Crescent Steel Co



Diam. of Pin Hole 2" | Diam. of Pin Hole 2"
Elong. of Pin Hole ____ | Elong. of Pin Hole ____
Nominal Section 2" × 1" Actual Section 2.01 × 1"
Original Area 2.01 Fractured Area 1.748
Gauge Reading for Elastic Limit 540 = 140.400 Pounds.
Gauge Reading for Ultimate Strength 910 = 236.600 Pounds.
Elong. in 6 Inches 7/16" Elong. in 2 1/2 Feet 7/8"

ELASTIC LIMIT 69.850 POUNDS PER SQUARE INCH.
ULTIMATE STRENGTH 117.710 POUNDS PER SQUARE INCH.
ELONGATION IN 6 INCHES 7.24 PER CENT.
ELONGATION IN 2 1/2 FEET 2.85 PER CENT.
REDUCTION OF AREA AT FRACTURE = 13.03 PER CENT.

Fracture Fine, close grain

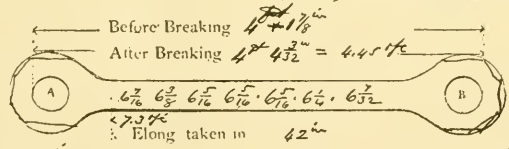
REMARKS

Carbon, Abt. 1.40
Other elements practically the same as No 1

TEST No. 3.

FULL SIZED TEST OF 2" × 1" Steel EYE BAR,

Rolled by _____ Manuf'd by Crescent Steel Co.



Diam. of Pin Hole 2" | Diam. of Pin Hole 2"
Elong. of Pin Hole 1/16" | Elong. of Pin Hole 1/16"
Nominal Section 2" × 1" Actual Section 2" × 1"
Original Area 2 0/2 Fractured Area ____
Gauge Reading for Elastic Limit 500 = 130.000 Pounds.
Gauge Reading for Ultimate Strength 960 = 249.600 Pounds.
Elong. in 42" Inches 2" Elong. in ____ Feet ____

ELASTIC LIMIT 65.000 POUNDS PER SQUARE INCH.
ULTIMATE STRENGTH 124.800 POUNDS PER SQUARE INCH.
ELONGATION IN 42" INCHES 4.75 PER CENT.
ELONGATION IN ____ FEET ____ PER CENT.
REDUCTION OF AREA AT FRACTURE = ____ PER CENT.

Fracture Broken in neck, slight flaw Close fine grain

REMARKS:

Analysis, practically the same as No 1 except carbon

Carbon .96

It will be noted that the .96 carbon gives the highest ultimate. This is in accordance with all of our experience, that steel of carbon .96 to 1.05 is the strongest in all tests except compression.

The absence of ductility in these tests indicates that they are above the safe limit for structural work.

A standard specification for crank pin and side rod steel is :

"Test piece should have a tensile strength of 85 000 lbs. per square inch, and an elongation of 15%.

"Steel will not be accepted if tensile strength is less than 80 000 lbs., nor if the elongation is below 12%."

Carbon.	Pounds per square inch. Elongation limit.	Tensile strength.	Elongation.		Reduction of Area. Per cent.
			In 2 ins.	Per cent.	
.50	63 560	84 220	.50	25	29.91
.60	71 500	108 800	.29	14.5	13.55
.70	69 980	117 400	.23	11.5	8.59
.60	.....	92 650	.28	14	17.32
.60	Slight flaw in fracture.	73 220	.44	22	23.91
.60	.....	81 160	.40	20	26.58
.60	.....	96 800	.30	15	16.26
.60	.....	97 170	.36	18	18.25
.60	.....	94 520	.36	18	16.96
.60	67 980	81 460	.36	18	26.24
.60	78 760	81 500	.26	13	12.56
.60	75 300	111 700	.42	21	27.27
.60	72 390	93 060	.40	20	27.22
.60	75 300	88 460	.40	20	21.88
.60	77 000	93 920	.40	20	20.85
.60	81 940	92 020	.26	13	
.60	75 650	86 760	.26	13	
.60	.....	100 000	.30	15	
.60	.....	104 100	.25	12.5	
.60	.....	93 620	.38	19	
.60	51 300	78 170	.36	18	25.09
.60	47 140	77 440	.42	21	29.39
.60	.....	93 430	.34	17	23.84
.60	.....	94 920	.35	17.5	27.37

We made three samples, .50, .60 and .70 carbon, respectively ; the tests are the first three in the table.

We adopted the .60 carbon grade, and gave the result of 21 tests, representing several hundred forgings. When the forgings were first made we found that the tests were a little high ; to correct this we annealed all of the forgings slightly, and the difference in the amount of the annealing accounts for the wide variations in the tests.

These forgings have given general satisfaction, and when the strains and shocks they are subjected to are considered, it would seem as if structural steel of the same quality might be safe as high as .50 carbon.

The mean analysis of these forgings is :

Silicon	.025	Highest	.046	Lowest	.009
Phosphorus	.016	"	.018	"	.009
Sulphur	.028	"	.05	"	.021
Manganese	.325	"	.45	"	.26
Carbon	.60	"	.65	"	.56

To test the effect of cold drawing, tempering, annealing and hardening and tempering, we took a piece of $\frac{1}{2}$-in. round cold drawn wire and cut it into four pieces.

Cold Drawn Bar.	Elastic Limit. Pounds per square inch.	Tensile strength. Pounds per square inch	Elongation.		Reduction of Area. Per cent.
			In 3 ins.	Per cent.	
Cold drawn broke in grip	92 420	141 500	.06	2.00	2.42
Same bar drawn black	114 700	138 400	.18	6.00	12.45
Same bar annealed	68 110	98 410	.30	10.00	11.69
Same bar hardened and then drawn black	152 800	248 700	.25	8.33	19.7

Analysis.

1.15 carbon.

Silicon	< .02	Phosphorus	< .02
Sulphur	trace	Manganese	< .30

The upper test is of the piece without treatment.

The next piece was heated until it passed through all of the temper colors to black, and then cooled slowly.

The third piece was annealed carefully.

The fourth piece was hardened carefully and then drawn through all of the color tempers to black.

The analysis is given below.

The figures speak for themselves.

G. Leverich, M. Am. Soc. C. E. (by letter).—The futility of an engineer in his specifications, prescribing conditions not of necessity correlated, is apparent ; thereby he, perhaps unconsciously, exposes a lack of the knowledge upon which his requirements always should be based. As well try to solve an equation of two variables ; impossible if they are independent, and to be done only when their relation to each other is known, and one drops out as unnecessary.

Usually he should not determine the methods whereby his materials are to be produced ; since he is unlikely to be as well informed concerning them as is the steel or iron maker, whose skill and judgment are at his command ; and if he were, the latter confined to a particular formula or process, is thereby deprived of that liberty of choice

and action which will best permit the exercise of his riper and more definite experience and secure a better product. Instead, the engineer should determine results ; by physical tests, he may establish beyond question or doubt, either on his or the contractor's behalf, the precise quality of the material supplied, irrespective of the kind and proportions of its constituents, and which the practiced chemist—much more the engineer, who confessedly is not such an expert—may err in detecting.

Generally, the engineer should deal with that which he knows, and leave the other pertinent matters to those who know better than he ; thereby he will tend to secure confidence in his judgment and prompt acceptance of his decisions. The old rule is still in force : "To do as one would be done by." An engineer has no right, neither is it politic for him, to insert in his specifications, conditions to govern the contractor which, if their positions were reversed, he would be reluctant or unable to observe.

As pertinent to the consideration of material differing widely in physical characteristics, and subject to similar stresses, the following is submitted, deduced from a record *more full than heretofore published, of tests made at the Watertown Arsenal twelve years ago of nine full-sized eye-bars for the lower chords of the Bismarck Bridge, when steel as a material for such members was in less favor than now.

These bars were 25 ft. long; they varied in width from 6.36 to 6.65 ins., in thickness from 0.98 to 1.62 ins., in transverse section from 6.49 to 10.77 sq. ins., and in carbon contained from 0.09 to 0.559%; per square inch of section, their elastic limits varied from 32 540 to 43 310 lbs., and their ultimate strengths from 46 600 to 74 890 lbs.; their extensions, per 200 ins. in length, varied from 0.119 to 0.1337 ins., under a stress of 15 000 lbs. per square inch; from 0.2445 to 0.2797 ins. under a stress of 30 000 lbs. per square inch, and from 4.37 to 37.25 ins. under their several breaking strains.

Diverse as these bars were in section and quality, as thus shown, still having reference to their extensibility as noted, had they been placed side by side to form a single lower-chord member and the same subjected to an aggregate stress of 15 000 lbs. per square inch, the stress on each would have varied from but 590 lbs. below to 960 lbs. above it; and had the aggregate stress been 30 000 lbs. per square inch, this variation would have been but from 1 150 lbs. below to 2 660 lbs. above it.

* "The Bismarck Bridge," a Report by George S. Morison, Engineer, 1884. See Appendix K and L.

From Record of Tests to Determine the Tensile Strength of Nine Full-Sized Steel Eye-Bars, for the Bismarck Bridge, Northern Pacific Railway, made at Watertown Arsenal, January 28th to February 4th, 1882, for George S. Morison, Engineer in Charge.

The bars were of Open-Hearth Steel and the Eyes were Made by the Kloman Process. The Gauge Length to Determine Elongation was 260 ins. The Pin Holes were 5 ins., and the Pins used were 4.98 ins. Diameter.

Dimensions—Inches.			Percent. of carbon.	Strength per Square Inch—Pounds.		Extensions in 260 Ins.			Stress on each bar, if the whole formed one lower chord member and were subject to an aggregate stress per square inch of—	
Width and thickness.	Section.	Length, center, to center of pin holes.		Elastic limit.	Ultimate.	15 000 lbs. strain.	30 000 lbs. strain.	Under breaking strain.	15 000 lbs.	30 000 lbs.
6.62 × 0.98	6.49	300.06	.09	33 900	59 580	.1321	.273	36.23	14 730	29 370
6.55 × 1.13	7.4	300.4	.1285	33 510	59 730	.1292	.272	34.78	15 060	29 470
6.54 × 1.28	8.37	299.8	.178	36 798	66 070	.119	.2455	28.13	15 960	32 660
6.6 × 1.02	6.73	300.08	.18	32 540	60 540	.1331	.2797	26.05	14 610	28 660
6.6 × 1.27	8.38	299.9	.183	32 900	63 240	.132	.28	37.25	14 740	28 630
6.56 × 1.05	6.89	300.75	.19	34 250	46 600	.135	.2779	6.17	14 410	28 850
6.65 × 1.62	10.77	300.29	.203	36 398	59 545	.1267	.251	14.1	15 530	31 940
6.5 × 1.21	7.87	299.9	.222	37 330	62 640	.1279	.2655	21.7	15 210	30 200
6.36 × 1.14	7.25	300.4	.559	43 310	74 890	.1337	.2778	4.37	14 550	28 860

A. C. Cunningham, Assoc. M. Am. Soc. C. E.—Realizing the incompleteness of the investigation in my eye-bar test, it was with some hesitation that I presented the case to the Society. Believing, however, that a few facts are better than none at all, I decided to submit what little data I was able to obtain through my personal exertions.

The thorough and comprehensive discussions of this case seem to have warranted my venture. What we are after, as individuals, and as a Society, is the truth, and these discussions have brought out a great deal of it.

This eye-bar was of steel, made by a reputable and experienced manufacturer, and was headed and tested by another equally experienced and reputable. Are either or both of these parties to be blamed for this bar? In my opinion they are not. The fault lies in the condition of things as they exist at the present time, and may be laid to incomplete and insufficient specifications, lack of information about steel on the part of engineers, and their difficulty in obtaining the same, and careless, incompetent or underpaid inspection. The conditions are not a little helped by the general desire to obtain a first-class article for one-half its value.

Before proceeding farther, I will ask your indulgence while we go over the generally accepted laws of segregation. In the first place, segregation means "separating out."

When steel cools from a melted condition, some of the mixed or combined elements have a tendency to separate from the hardening metal and remain in the more fluid portions, in the same manner that freezing water has a tendency to free itself from foreign substances.

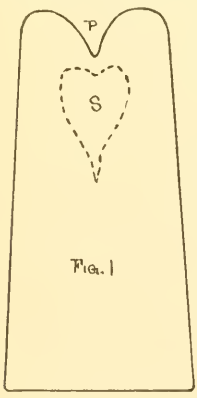


Fig. 1

As the steel continues to harden, this separation goes on until the portion which hardens last will contain the greatest amount of the separating elements.

Among the elements which have a tendency to separate from the hardening steel are carbon, phosphorus and sulphur, while manganese does not have much of a tendency. Thus, three of the most important elements of steel, two of which are the most dangerous, are certain to be unequally distributed throughout an ingot. This variation is not infrequently double between the greatest and least segregated parts.

Let us now consider a normal ingot (Fig. 1).

This ingot shows a pipe at *P*, and we have reason to believe that the greatest segregation exists in a pear-shaped space as indicated by the dotted line about *S*. It is readily seen from this figure how an ingot can be sufficiently cropped to remove all piping and still retain the maximum segregation.

I note in the discussions that the opinions are divided as to the desirability of excessive cropping, to remove segregation. The matter can be arranged otherwise, if deemed advisable.

The fast-extending basic process can be called upon to dephosphorize and desulphurize to such an extent that the maximum segregation which can take place shall be harmless. This will lessen the danger from segregating carbon, and, in the meantime, we can search for a new hardener to replace carbon, which does not segregate.

Where acid steels are more desirable, the basic process again will serve to furnish a melting stock for them, so that the final acid steel may be as free from dangerous segregation as the basic.

Attention and study of the size and form of ingots, their rate and manner of cooling, and the casting temperature of the steel, will all lead to the prevention of excessive segregation. I am not aware that these things have received any general amount of attention yet, nor do I think they will until we have insisted upon the excessive cropping, or competition forces the matter.

There can be no valid objection to an engineer specifying limitations to the dangerous elements of steel. Neither should there be any objection to his specifying limitations to all the known elements of steel, provided a reasonable variation is allowed.

Before the introduction of manganese, steel made but little advance, and while we know that this element is a good thing, we have also learned that too much or too little of it may be very bad under certain conditions.

The engineer who has specified chemical limitations on his steel, though better off than the one who has not, must not feel too assured of his safety.

As specifications are generally written, read and translated, the chemical requirements are determined upon the manufacturer's "heat" test. This is a small sample dipped from the still melted charge and which chills so quickly that little segregation can take place; in addition, average drillings may be taken from this sample. This "heat" analysis fairly represents the average condition of the steel, but it is certain that in every ingot there will be steel containing more and also less of the elements than are shown by this test.

Too much reliance has so far been placed upon tension tests alone. I do not question their value, nor would I have them abandoned, but I think it is nearly time for them to step into the rear rank.

When a certain grade of steel is ordered from the manufacturer, he must take into account the chemistry of his stock and the chemistry of his finished steel, and, with certain allowances for work and rolling, his material, in normal cases, fills the physical requirements. From the known analysis he can calculate the ultimate strength within a few thousand pounds, and for him the tension test becomes simply a check.

The engineer who desires a true and thorough knowledge of his steel must follow it from the raw to the finished material. From time to time we hear of new and simple tests to be applied to steel which are to tell us all about it without other aid. Such things must be regarded with caution, for, although they have their value, a true knowledge of steel can not be derived without its comprehensive consideration.

Mr. Roberts' questions indicate an investigation and inspection which would have given us a very satisfactory insight into this case. Unfortunately it was impossible for me to carry the matter out on these or similar lines.

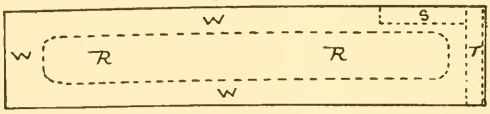
Let me ask—how many engineers have all the information about their eye-bars that is indicated by these questions ?

The general adoption of the chemical limitations suggested by Messrs. Roberts and Blakeley would be a great advance in specifications; they would also practically prohibit the use of Bessemer steel.

I will ask you to note that I do not assert that this analysis is due to segregation; as Mr. Christie suggests, it may have been the general character of the steel. If it was, an examination of the segregated portions would have been an interesting study. The high tensile stress developed in the second testing I think is a proof of the care that was used in forming the second head.

I agree with Mr. Macdonald that the quality of eye-bar material should be definitely settled before the material leaves the mill, and it could be; unfortunately, it is not. Fig. 2 will illustrate this, which

Fig. 2.



we will assume, for an extreme case, to be a 10 x 2-in. bar made on universal rolls. These heavy sections consist of a well-worked shell *W*, surrounding an unworked or raw core *R*, which is coarse and open in its nature, and may be highly segregated.

The specimen tests are taken as at *T* or *S* and represent the well-worked shell and ignore the unworked center; neither will an analysis from them indicate any segregation.

The Hawkesbury specifications would be excellent if we knew all about our steel to start with. As it is, they have the inherent defect of taking no account of the different melts of steel, and it is not an impossible supposition to say that every bar might be from a different melt.

I would call attention to my statement that after the first head broke, there was no elongation or reduction in the body of the bar.

The elastic limit reached was probably at the opposite end of the bar from that in question, and there could have been no serious distortion of the material before the second head was formed. The second head broke in solid material, and not in the flaws, as was expected.

The proper way to restore strained steel is generally considered to consist in annealing it, or heating and reworking it. This bar had both.

Mr. Morison's idea of discarding defective bars, I think, is a mistake. It is all right to require a perfect bar for an official test, but I believe, in addition, that a specification should require the testing of every bar that is defective.

It is from the study of known and visible defects that we arrive at a knowledge of the unknown defects, and will finally learn how to make perfect bars.

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703.
(Vol. XXXI.—April, 1894.)

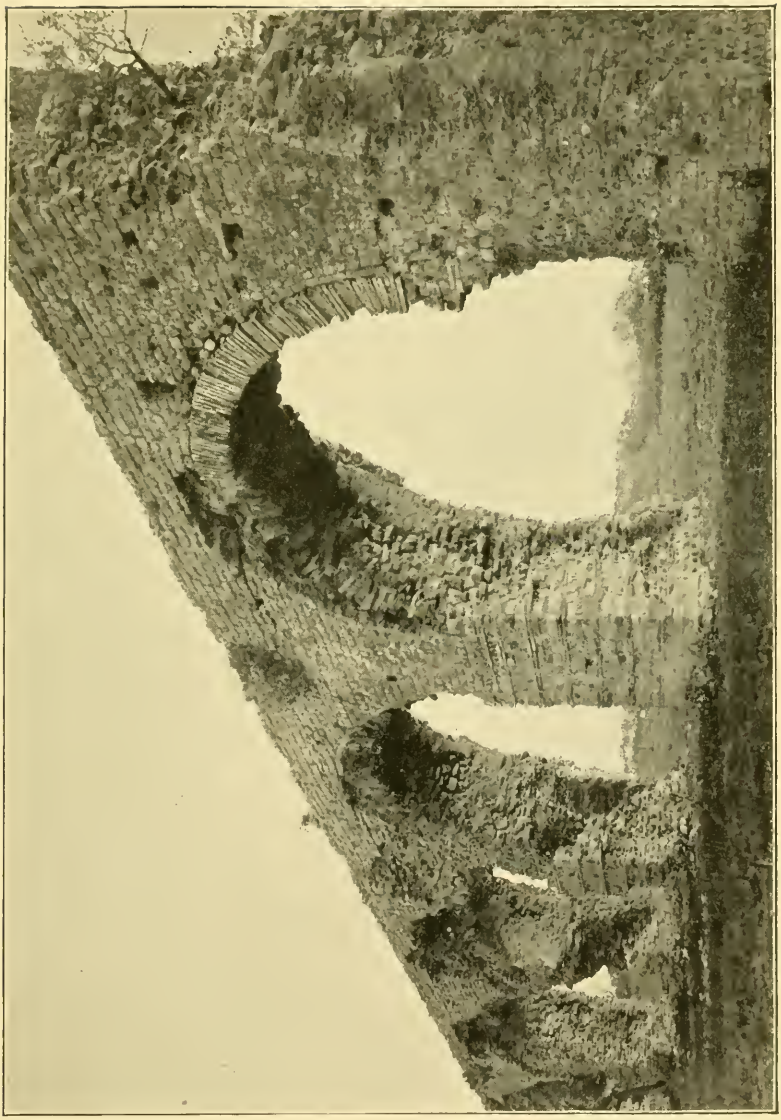
THE DEVELOPMENT AND RECENT IMPROVEMENT OF CONCRETE-IRON HIGHWAY BRIDGES.

By Fr. von Emperger, C. E., M. Austrian Society of Engineers and Architects.

Read April 4th, 1894.

WITH DISCUSSION.

The combination of concrete and iron was first used about 16 years ago for building purposes, and it seems to the writer very characteristic that the first use of concrete in bridges, both with and without iron, was begun in the country where the remains of a number of old concrete bridges show that the old Roman "engineers" had something to teach to posterity. It is very likely that this standing example of engineering skill must have impelled the French engineer to works which could be called a "renaissance" in bridge work. They could accomplish more, as the Portland cement of our days is a better material for the purpose than the pozzuolana, which the Romans used in their structures. Some of the latter are still entirely untouched in their exterior finish; as, for instance, the Pont du Gard near Nimes, France, and do not show the interior construction of Roman concrete. The accompanying illustration (Plate LIII), which represents the Aqueduct of Vejus, however, shows that the Roman concrete had larger stones than we use now, but laid without dressing in a bed of mortar and gravel.



While the use of concrete for bridges soon became general in the central part of Europe, its application was not common in other countries and advanced slowly with the progress of the manufacture of Portland cement.

In this country, for instance, the use of concrete has been almost confined to floors, and the writer would especially mention, Phineas Ball, M. Am. Soc. C. E., of Worcester, Mass. (1884); Mr. Ransome, of San Francisco (1884); and Mr. N. Poulson, of New York (1892), as among Americans who have improved and used concrete with iron in floor construction. The number of concrete bridges is very small, and the writer knows of but three concrete-iron bridges in this country. In view of this fact, he believes that some of the experience had in Europe with this combination will interest American engineers, as well from the historical as from the engineering point of view.

The man to whom the invention of this combination is generally attributed, is Jean Monier. He lived at the beginning of his peculiar career (1876) near Paris, and was in the gardening business. His intention was to build large and strong tree-pots, as they are much used there. Monier used for this purpose concrete with imbedded wire nets, and invented the so-called "Monier System." We see that he was far from being a trained engineer, but had an inventive head, which often is worth more than training. It is idle to state that he was not the discoverer of the process; but he perceived the value of his method, developed it with the help of trained engineers and capitalists, and, what is always the most important, he made a success of it.

We hear first of him when he received a 1st class Diploma of Honor at the exposition in Paris (1878), and, in a comparatively short time, he built nearly 1 000 water and gas tanks throughout France, some of them 70 ft. in diameter and 11 ft. high. The next application of this system was to tubes and sewers; then came vaults and bridges, and finally flat floors.

In France and Germany, where the use of concrete for bridges was common, even before that time, the application of this improvement found its best field, as in those countries it was not necessary to create public confidence in the method, and builders were easily made familiar with this application.

This is, however, the serious difficulty which has to be overcome in other countries.

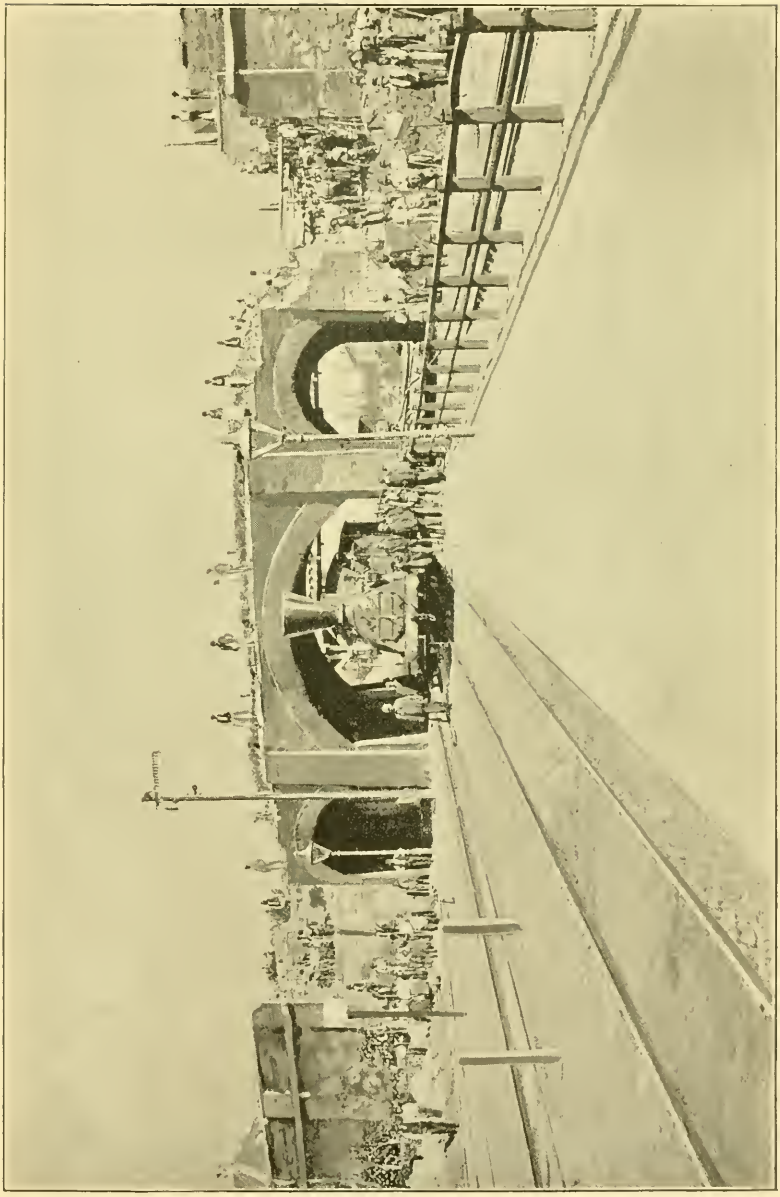
It is, therefore, not out of place to remember that quite a number of concrete structures, in addition to the Roman water conduits, have stood through centuries; as, for instance, the cross-arched vaulting (78 ft. span) over the bath of Diocletian, in "Hexædra amplissima," and the parabolic main aisle of St. Peter's (87 ft. span and 150 ft. rise), both in Rome.

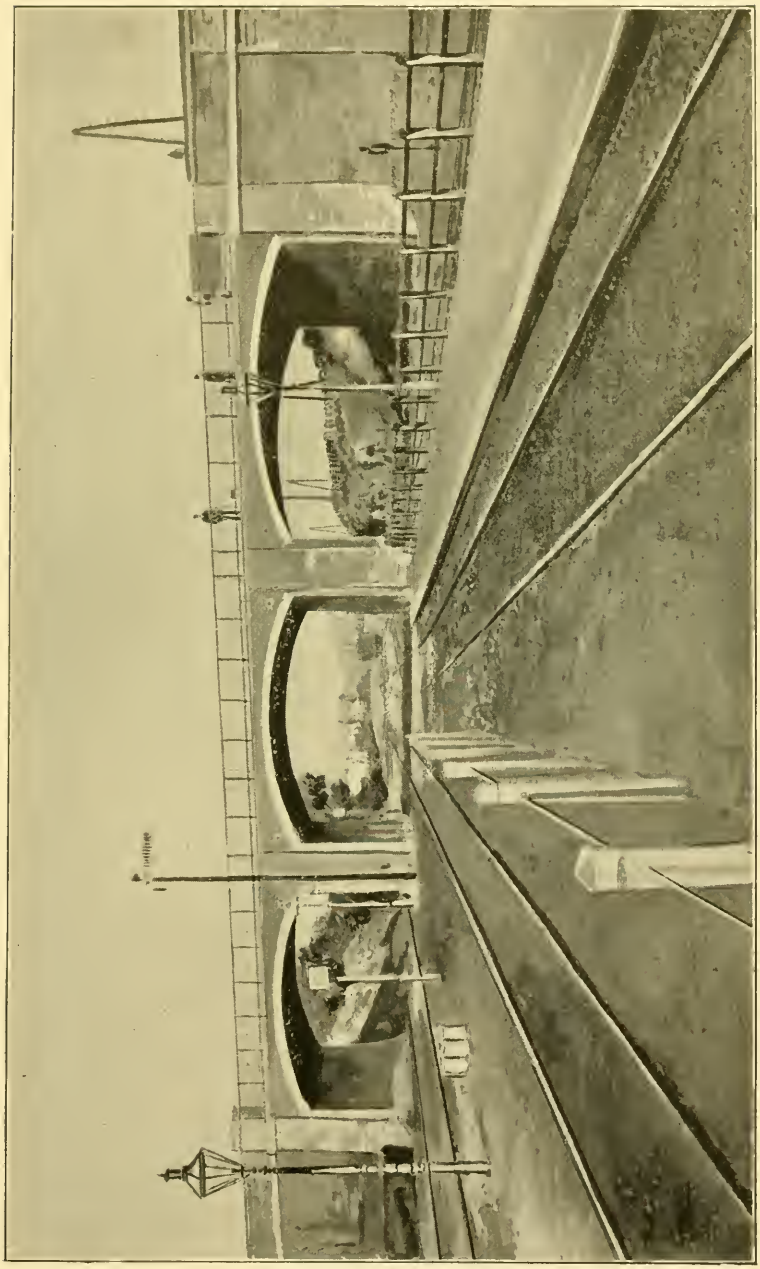
Speaking of modern structures, the writer may mention the bridge over the Isère, France (span, 85 ft.; rise, 8.5 ft., and 2.5 ft. thickness in crown); the bridges over the Saevale and the Iregua near Torio, Spain (spans, 33 ft.; rise, 7.5 ft., and 3 ft. thickness in crown), and the aqueduct at Vannes, France, built in 1868–69.

The last-mentioned structure has been described in General Gillmore's excellent book, "Coignet Béton." The arches have 75 and 125-ft. span. Later on, the Pont d'Alma and Pont Napoléon were built as railway viaducts near Paris, spans about 140 ft., and one-tenth rise. Another more recent example (1873) is a bridge of 105 ft. span, 13 ft. rise and 20 in. thickness at the apex, near Erlach, Germany. Its designer, Highway Inspector Koch, provided it with three joints, of asphalt plates, so it was turned to a three-hinged arch —an example which has been repeated later several times with lead plates or ball joints. But it seems notable that at this time (1873) any one could have the faith to build such an arch on soft ground. The arch settled, as provided, 5 ins. without injury. It is impossible in the limits of this paper to continue the line of examples of concrete bridges, but the writer may be allowed to mention the bridge built by Dr. Leube, near Ulm, Germany (1885), on account of its bold dimensions. The span is 150 ft. and the thickness in crown is only $6\frac{2}{3}$ ins. Such figures surprise even the engineer, for if these bridges show their proportions plainly, our eyes have to grow accustomed to these delicate forms.

We often hear it praised as great progress that, by the use of Portland cement mortar, stone bridges can be reduced in size 10 per cent. Experience in Germany has proven, however, that with concrete and its monolithic properties a saving of 25% is attainable. A stone arch must be constructed for pressure only; in concrete, however, one-tenth of the pressure (Engesser says one-seventh) may be taken up as tension limit. To illustrate the difference between masonry and concrete-iron construction Plates LIV and LV are given, and show the highway

PLATE LIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 703.
VON EMPERGER ON CONCRETE-IRON BRIDGES.





bridge over the railway station at Moedling, near Vienna, as it was originally built, with masonry arches, and as it is now (after 1889), with the Monier system.

The leading idea of the use of iron with concrete is to strengthen the monolithic concrete block against tension, that the unavoidable waste of material involved by a low tension limit and a ten times higher pressure limit may be overcome, and to enable us safely to rely on the tension in the combination. This enables us to build flat as well as thin arches, saving 67% and more, in comparison with stone.

The genuine Monier system had only one wire netting on the intrados, but engineers soon became aware that this was not sufficient. Tension occurs, also, on the intrados at *B*, as well as in the extrados at *A*, Fig. 1.

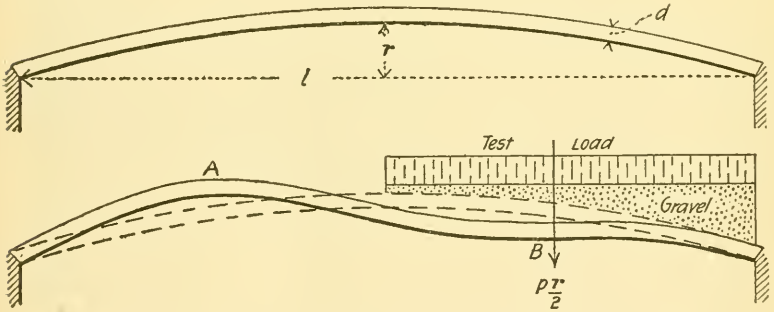


FIG 1.

Two nettings were, therefore, used in larger spans, as shown in Fig. 2, which is a reproduction of tests made by the Building Department in Berlin, Germany, 1886.

From the great number of tests made, to get the real value of this invention, may be noted those of G. A. Wayss & Co., the owner of the so-called "Monier patent," in Germany; also, those of the late Professor Bauschinger, in Munich (1887), and those of the Austrian South Railroad (1889), and others.*

All those tests clearly showed the superiority of this combination over any other arch construction of brick, stone or concrete alone.

As most of the prominent Monier bridges are known through the technical press, the writer only reproduces one (Plate LVI) which has a span of 132 ft., 14¾-ft. rise, and is 9⅞ ins. thick at the crown. It was

*See "Die Monierbauweise," by G. A. Wayss, A. Seydel, Berlin, 1891.

designed for a live load of 200 lbs. per square foot, but was built for show purposes only, in a fair (1890) in Bremen, Germany. It may be added that the whole work (concrete approaches and stairs) was finished in six weeks; the main span in 36 hours. The number of Monier

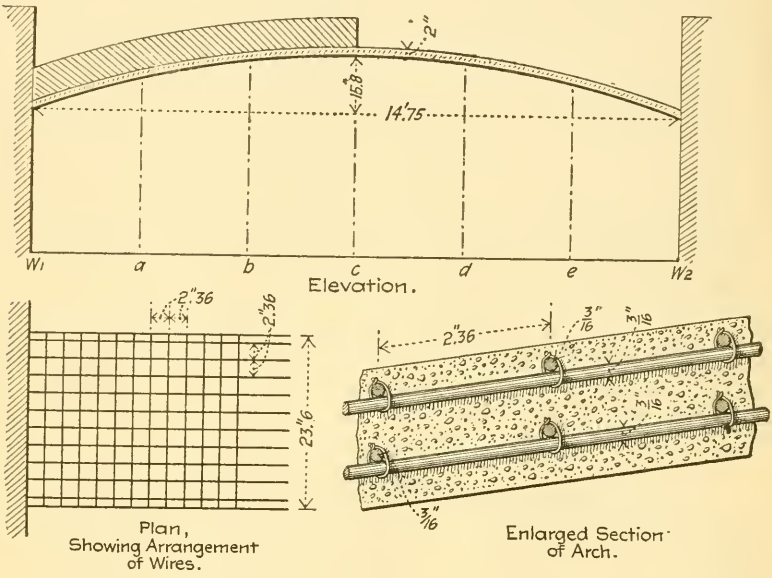
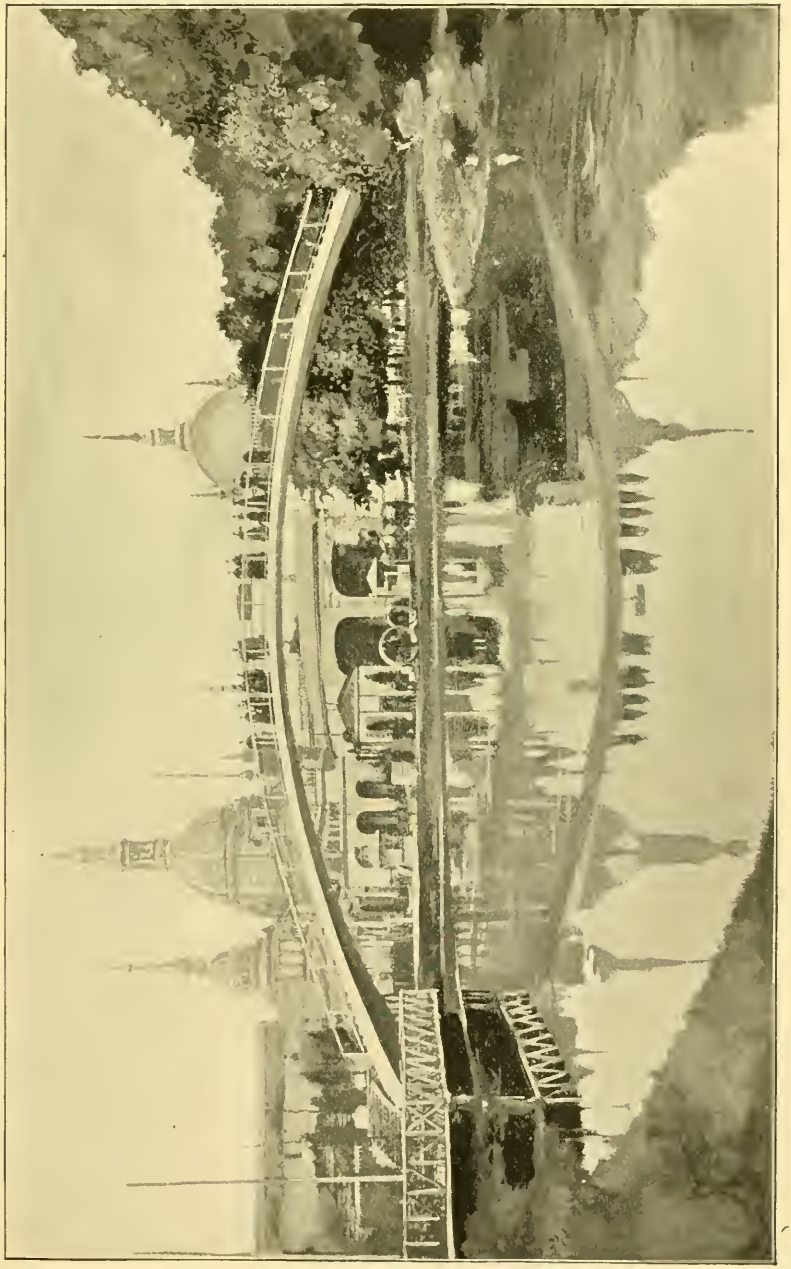


FIG. 2.

bridges in Europe is very large, and the writer's investigation outside of Germany and Austria is not complete. In 1891, some larger Monier bridges were built in Switzerland; one at Wildegg and two at Aarau. All three have 128 ft. span, 11 ft. rise, and are $6\frac{2}{3}$ ins. thick at the crown and 10 ins. at the abutments.

The iron netting used with the Monier system is not stiff itself, and its co-operation with the concrete can be expected only after hardening; but if the work is carefully done, this is of no importance. But what will happen if the work is not well performed? In common building practice it is not extraordinary to find a dishonest contractor who uses an improved kind of dirt instead of concrete, or who hurries the centering away to another job. There is no material which is not liable to failure under such circumstances. Such a case happened just in the beginning of the Monier system in bridge work; the arch fell without warning, and several men were killed. This failure caused



general distrust, and a remarkable set of tests was needed to show that it was not the fault of the system itself.

Nevertheless, this difficulty has been overcome by the other systems now in use, which use rolled shapes instead of wire netting.

Breaking tests have shown that any fault in the concrete must show by distinct cracks, and an overloaded bridge of that kind will go down slowly, as the stiff rods must break first. The only injured person in case of dishonest work will be the contractor. The inventor of this latter method is R. Wünsch, of Buda-Pesth, Hungary (1884). Mr. Wünsch has built with his system six highway bridges, all in Hungary, as follows:

Name.	Span—Feet.	Rise—Feet.
Belovar	41⅓	3
Nyeresfalva	33⅓	3⅓
Munkacs	13⅓	1⅓
Kaschan	27	3⅓
Ladomer	14⅔	1⅓
Neuhausel	55⅔	3¾

The first of these is shown in Plate LVII.

The last one has been fully described in the *Zeitschrift*, of the Austrian Society of Engineers and Architects, and reprinted by the American technical press. The principal used by Mr. R. Wünsch is shown in Fig. 3, which represents a test made with a 16.6-ft. bridge by the Hungarian government, and the results are given on page 444.

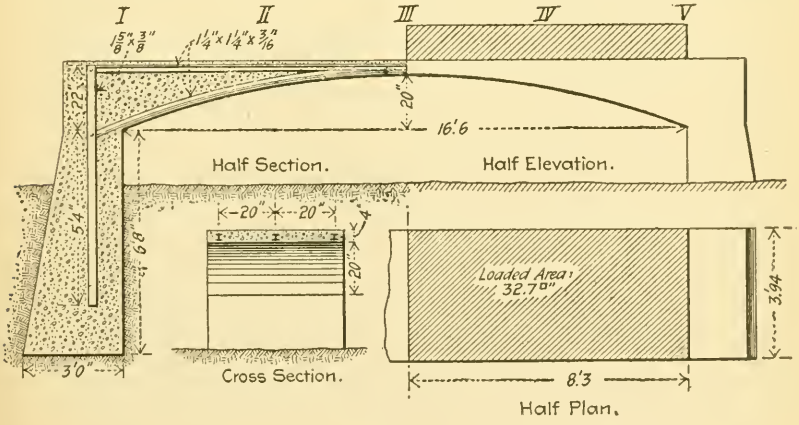


FIG. 3.

Total load.	Load per square foot.	Deflections at Point.		
		II.	III.	IV.
Pounds.	Pounds.	Inches.	Inches.	Inches.
2 844	88	....	....	0.04
5 049	105	....	0.04	0.04
8 356	258	....	0.16	0.16 *
11 673	360	0.04	0.27	0.27 †
16 082	470	....	0.59	0.55 ‡
18 287	565	0.75	2.64	1.97 §
33 101	1 010	....	15.75	....

The structure was designed for a live load of 80 lbs. per square foot.

Two points in every section were closely watched, as well vertically as horizontally. There was no measurable horizontal motion of the abutment, and the inventor claims it as the most important property of this system, that the horizontal thrust is changed to a lift by the anchorage. This would enable us to place the Wünsch arch on top of a high pillar, as well as a flat girder.

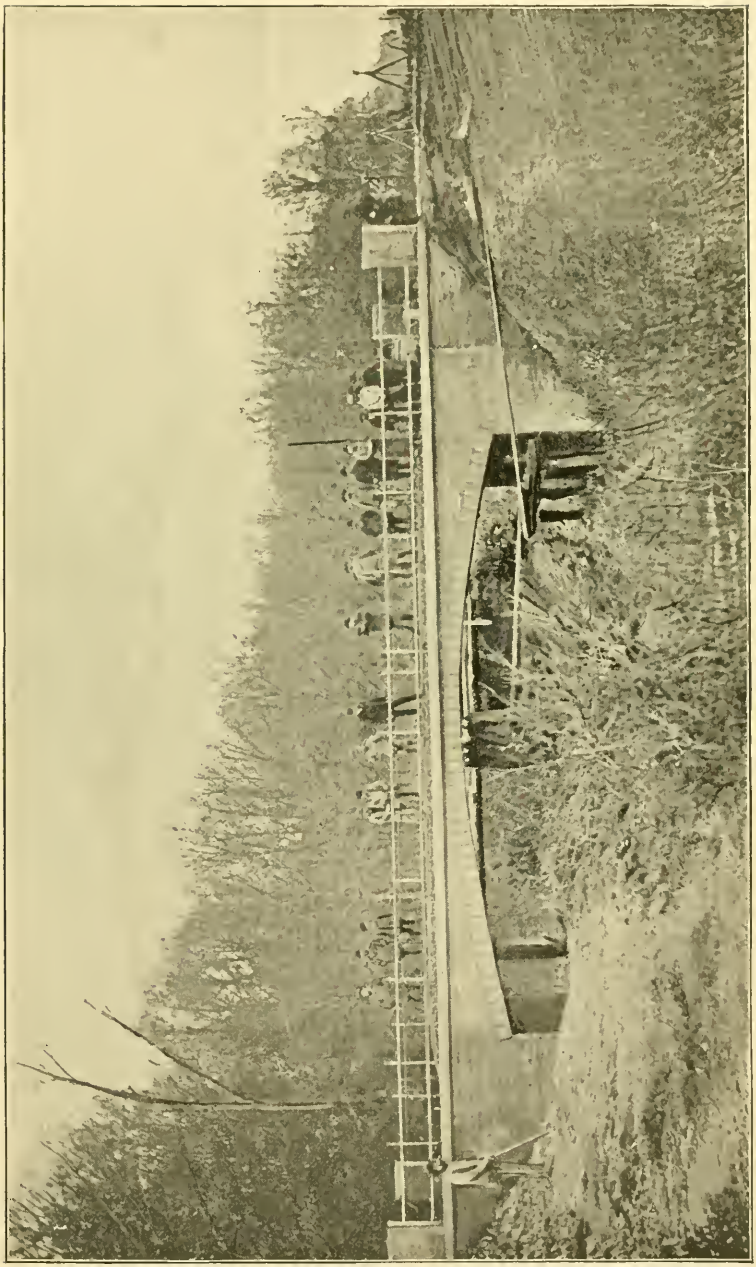
The latest improvement in this line is called after the inventor, "System Melan," Fig. 4. It consists of bent I beams, with a concrete body between and no cross-rods at all. It is a very remarkable saving of iron, and, at the same time, an advance in strength. The beams are, of course, the stiffest shape to be had. They are a kind of centering itself. It is not difficult to show that these bent beams would safely carry alone the load intended, and it is hard to understand why this simple construction has not been thought of before. Simplicity in all details is a property which will certainly be appreciated in this country. No strain sheets, no worked-out plans, are needed. All that we must know are four figures—span, rise, section of beam and distance between. The bending is done in the mill at a cost of ½ cent per pound or less, and there are no other connections but splices to prolong the beams. All that the workman has to look after is to keep the beams flush. After these are once in place, the centering can be finished in accordance with the given line. The centering need not be heavy; it has not to support the entire structure, as with other concrete systems,

* Between Sections I, II and IV, V, fine cracks.

† Crack unchanged.

‡ Crack open to 0.08.

§ Cracks increase slowly until the breakdown.



application is only one of economy; and that there will be places where cement and labor can be had so cheaply that there is no saving in using iron. On the other hand, there are locations where sand and gravel cannot be had within reasonable distance, and an iron bridge must be advisable under any circumstance. Such a question must presuppose average circumstances.

The Austrian Society of Engineers and Architects undertook a very important step to get at the merits of the existing system of arches. As a measure of the importance of these tests, the fact may be mentioned that they cost 20 000 florins, while the work done by manufacturers without pay had a value of 15 000 florins. The writer's experience in the commercial values of the two countries enables him to say that these tests could be repeated here at an expenditure of $36 000.

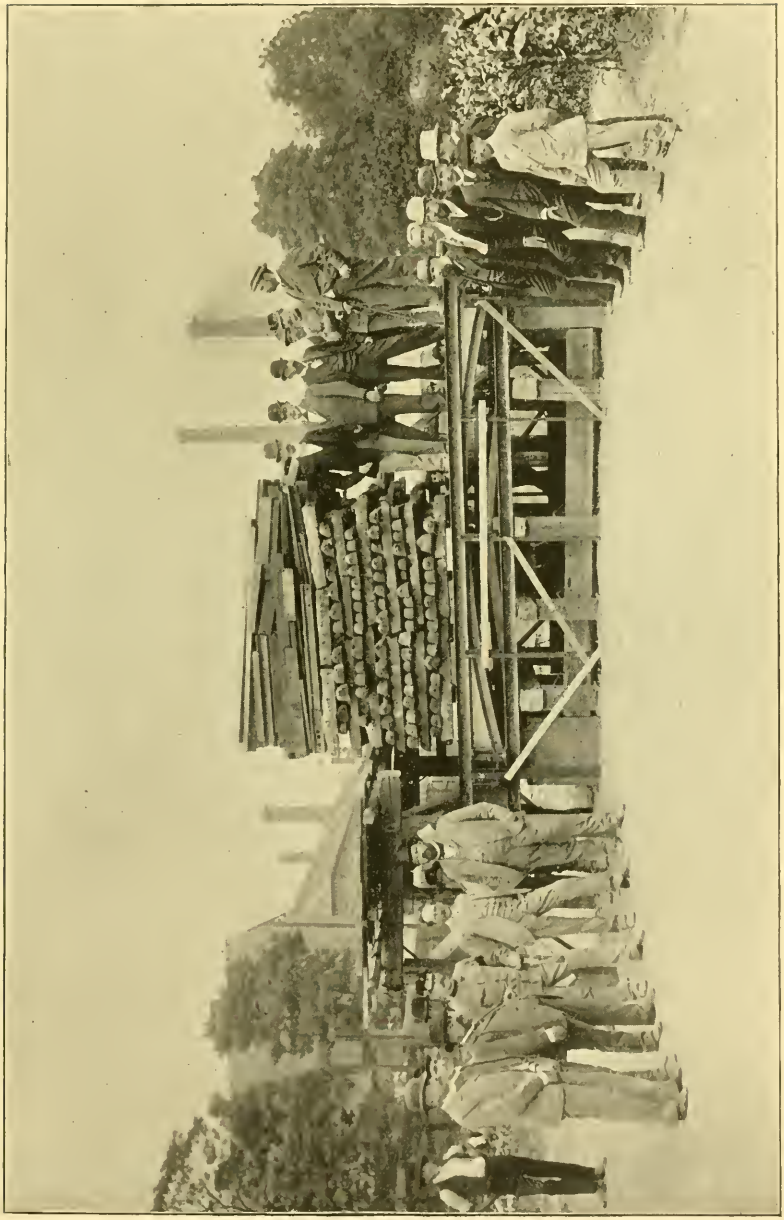
They built arches of all kinds and sizes (from 7 to 77 ft. span), loaded them and broke them down, to get the elastic dislocation and breaking loads. The tests were superintended by a board selected from the principal railroads, builders and colleges, whose duty it was to see that the arches were not built either better or worse than in common practice.

In the following table the four first tests are from this series, while the last one was made by the Hungarian government (1891), as above stated :

	Material.	Span. Feet.	Rise. Inches.	Thickness in crown. Inches.	Steel. Pounds per square foot.	Breaking load. Pounds per square foot.
Fig. 1.....	Brick with cement mortar.....	13⅓	15¾	6	....	321.5
Fig. 1.....	Concrete.....	13⅓	15¾	4	....	737.3
Fig. 1.....	Monier.....	13⅓	15¾	2¾	4	839.7
Plate LVIII.	Melan	13⅓	11.0	3⅛	1.4	3 360
Fig. 3.....	Wünsch	16.4	20.0	4	1.5	1 010

All arches were loaded as shown on Fig. 1. The Monier arch had one wire net $2\frac{3}{8}$ x $2\frac{3}{8}$-in. meshes out of $\frac{3}{16}$-in. rounds (see Fig. 2), while in the test of the arch of 77-ft. span, referred to above, two wire nets were used.

The Melan arch, $3\frac{1}{8}$-in. I beams, 4.3 lbs. per foot, and 40 ins. apart, and the structure was one year old when tested.



The Wünsch arch had $1\frac{1}{4}$ x $1\frac{1}{4}$ x $\frac{3}{16}$-in. angles with the poorest concrete, and was tested after one month, which explains its lower figures (see also the record of its load, which has been given before).

The test arch, System Melan, is shown in Plates LVIII and LIX. It was designed for a total load of 200 lbs. per square foot, and nobody expected so high a breaking load. Plate LVIII shows 1 500 lbs. per square foot. As no more pig iron was at hand and a margin of seven was reached, it was the intention of the Board to fulfill the programme and to break the arch down.

It was decided to load only one rib in the most dangerous manner by a square of 10 sq. ft., as can be seen in Plate LIX, which shows the broken-down arch.

The first cracks appeared with 3 000 lbs. per square foot, and the arch broke slowly down under 3 360 lbs. per square foot. This is certainly a load which the $3\frac{1}{8}$-in. beam alone never could stand. The safe load of this I beam (flat span as above, $13\frac{1}{3}$ ft.), is 73 lbs. per foot run, and it carried 11 200 lbs. per foot run, or 153 times more; but if we compare it with the bent I beam of the same span we will see that this beam is strong enough to carry the safe load alone. The high degree of safety involves, without doubt, a waste of material, and it is very likely that in the future the stresses used in the calculation may be raised. On the other hand, in the writer's opinion, a coefficient of safety of 5 is not sufficient, and 10, as Trautwine uses with stone, is entirely warranted. The figures used in the arch calculation for tension of concrete vary greatly. This shows mainly, that it depends more upon the theoretic than on the actual stresses, and there is a future for some coming experimenter. Professor Engesser recommends in accordance with tests of concrete arches made by Dyckerhoff & Widman, the use of 213 lbs. per square inch. But this figure leads to paper-like dimensions. Mr. A. Rella recommends 125 lbs. per square inch; while J. Melan, in connection with his calculation given further on, uses 60 lbs. per square inch. The latter figure is certainly too small. The breaking load, so obtained, shows a coefficient of safety of about 15.

The strength of concrete in tension can be taken from 290 to 340 lbs. per square inch. We see from this that Melan uses a theoretically warranted factor of safety of 5. But this proportion only applies to the limit of elasticity, and while we know that a concrete beam breaks under about double the theoretically calculated breaking load, we find

the figure of Professor Engesser explained if we suppose that an arch will break under treble the theoretically calculated load.

On this supposition Professor Engesser has a safety factor 5 times and Professor Melan one 15 times that of real safety, as stated. It would exceed the limits of this paper to say more about this matter; it seems sufficient to add that for the cement mortar commonly used with concrete in bridge work (1 to 3) it is necessary to specify a tensile strength of at least 200 lbs. after 28 days.

All this considered, we have to adhere to the calculation for concrete iron arches, of Professor Melan, as the best for the present until something better shall be at hand.

Calculation of Concrete Iron Arches.

We call—

The dead load in pounds per square foot $= q$;
" live " " " $= p$;
" span in feet $= l$;
" rise in inches* $= r$;
" thickness of concrete in inches $= d$, this generally equal to height of beams;
" distance between beams in inches $= a$; and further:

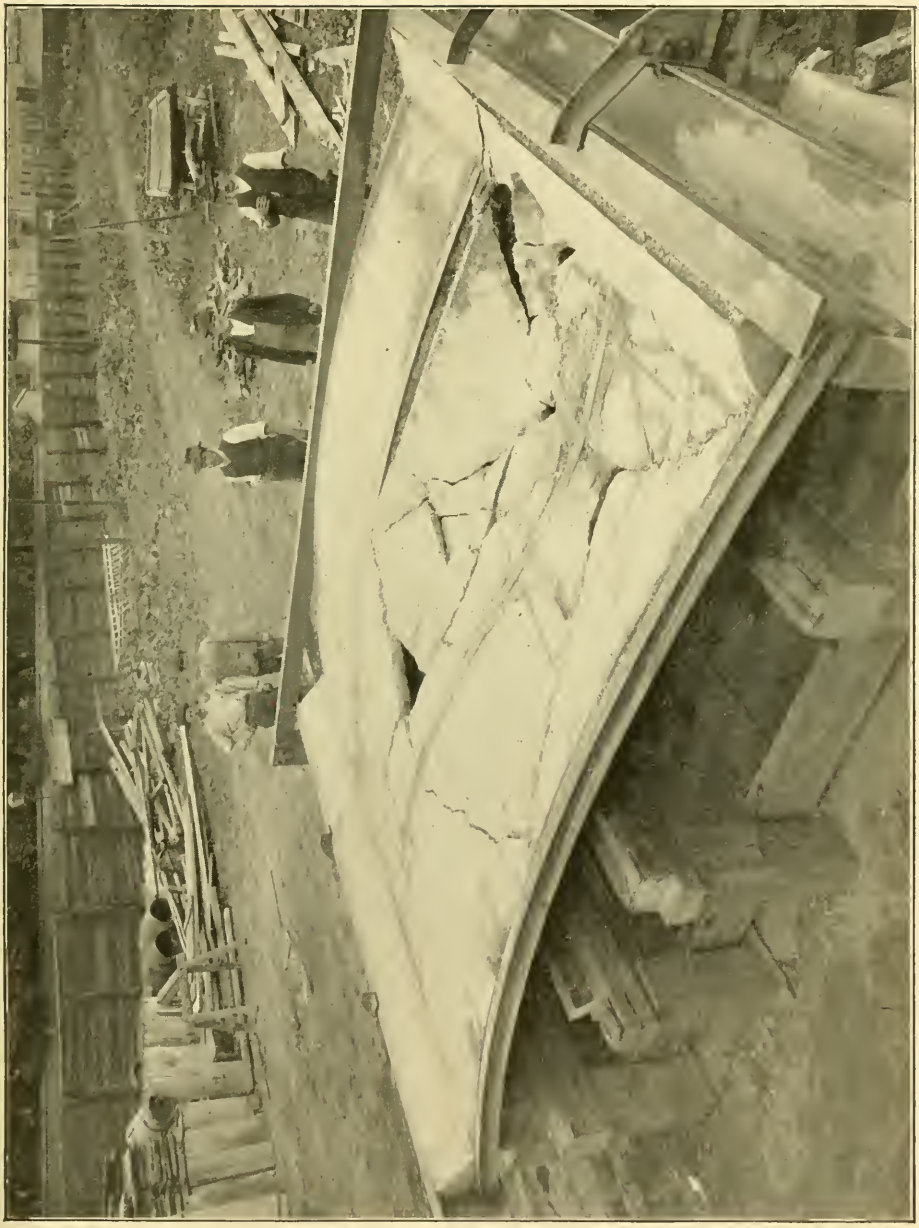
	Concrete.		Steel.	
Modulus of elasticity.....	E_1	750 000	E_2	30 000 000
Section.....	A_1	$a\,d$	A_2	
Moment of inertia.....	I_1	$\frac{a\,d^3}{12}$	I_2	
Moment of resistance.....	R_1	$\frac{a\,d^2}{6}$	R_2	
Coefficient of distribution.	α		β	

$$\alpha + \beta = 1.$$

Thesis.—The stresses in both materials, or, what is practically the same, the total loads L, can be assumed distributed, $L = L_1 + L_2$ if it answer the following proportion:

$$\frac{L_1}{L_2} = \frac{E_1 I_1}{E_2 I_2} = \frac{d^3 a}{480 I_2} = \gamma \ldots\ldots\ldots\ldots\ldots (1)$$

* The rise r is expressed in inches for simplicity only, the results desired being in square inches.



This put in—

$$L = L_1 + L_2 = \alpha L + \beta L = \frac{\gamma}{1+\gamma} L + \frac{1}{1+\gamma} L,$$

or

$$\alpha = \frac{\gamma}{1+\gamma} \text{ and } \beta = \frac{1}{1+\gamma} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The span and rise being given, we have to assume the size of beam d, and its distance a. The proportion $\frac{a}{d}$ is limited by the necessity that the concrete body has to act as a main and also as a cross-beam. It carries $L_1 = \alpha (q + p)$ as a main girder, and transmits $L_2 = \beta (q + p)$ to the iron beam.

With reference to the elastic theory, in cases where spans in two vertical plans occur, by reducing the allowed stress in concrete to 42 lbs. per square inch, we get—

$$\frac{a}{d} \leqq \sqrt{\frac{\frac{1}{3} \times 42 \times 144}{\beta (q+p)}}$$

$$\leqq \sqrt{\frac{8\,064}{\beta (q+p)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

If we find the a of our choice is not too large, we proceed to get the stresses in the main girders. A power H, which is not centrally applied to a section A, produces consequently a moment M, with the following stresses in the outer fibers :

$$S = \frac{H}{A} \pm \frac{M}{K}$$

we only have to use our signs tabulated above and we find the stress per square inch in the concrete :

$$S_1 = \alpha \left(\frac{H}{d} \pm 6 \frac{M}{d^2} \right) \quad \ldots\ldots\ldots\ldots\ldots (4)$$

and in the steel beam—

$$S_2 = \beta \left(\frac{H}{A_2} \pm \frac{M}{K_2} \right) a \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

It can be seen by a glance, that by proper choice of α and β, which mainly depend on a and I_2, the tension in the concrete or the pressure in the iron can be reduced to its proper limits.

To get the highest tension and pressure, we must consider as previously stated a half-loaded span (Fig. 1).

The horizontal thrust is equal—

$$H = \frac{1}{8}\left(q + \frac{p}{2}\right)\frac{l^2}{r_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

r_1 we call the rectified rise

$$r_1 = r + \frac{15}{16}\,\frac{d^3\,a + 12\;I_2}{(d\,a + A_2)\,r} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

The moment in a section $\frac{3}{16}\,l$ apart from the center is—

$$-M = \frac{9}{1\,024}\,p\,l^2 + \frac{1}{3}\,H\,(r_1 - r) \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

For a full loaded span—

$$H = \frac{1}{8}\,(q + p)\,\frac{l^2}{r_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Moment in crown—

$$M = \frac{1}{3}\,H\,(r_1 - r) \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Moment in abutment—

$$-M = \frac{2}{3}\,H\,(r_1 - r) \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

The last three equations are of use so far as the spandrels will be considered, while for ordinary use the equations 1 — 8 will be sufficient.

For the abutment itself we have to compute H from equation No. 9 with the weight of it, and to see that the resultant keeps within the inner third; and further, that the total weight has a proper area in the foundation.

Compared with other arch calculations this is extremely simple, and correct enough for practice as a comparison with an exact method, which we get by introduction of moment of inertia and the section in equation No. 1.

The calculation is, of course, not so simple as with a flat beam, especially as we are familiar with mill books. It is the intention of the writer, as soon as time will allow him, to tabulate the results in the same way.

On the other hand, if this calculation is once made, no elaborate plans or strain sheets, and only four dimensions, are needed, viz.: span, rise, beam and distance.

It should not be overlooked that in the foregoing calculation the modulus of elasticity of concrete (such a concrete as we use for bridges) is taken at 750 000. This is about half the figure we have been in the habit of assuming; see, for instance, General Gillmore and others who all give their figures with a great reserve, and some of

them directly doubting their correctness. This figure is the result of the experiments made by Professor Boek in Vienna, Austria, which were recently made in connection with the great undertaking of the Austrian Society of Engineers and Architects mentioned above, and is important as confirming the results of previous tests of beams of concrete alone, as well as in combination with iron.

This investigation was not confined to the arch alone, but extended also to the material used in its construction. The great amount of matter explains why the final report of the committee, although well known in engineering circles in Austria, has not been published yet. The writer is informed that it will appear next month, and it can then be found by those interested.

Two concrete structures which give the best proof of the elastic properties of this material may be mentioned. One is a chimney 160 ft. high, in Ireland, in one piece of concrete, which has stood the heaviest storms, and another example, which is cited by Mr. A. Rella, are wine tanks of a capacity of 80 000 galls. (of concrete only) in Agram, Hungary, which stood the last earthquake without cracking. A further proof is the use of concrete in railroad structures directly below the tie.

There is this excuse for a novice, that, in speaking of concrete, he thinks only of the glazed claypot, but if technical papers speak of concrete as "inelastic, like cast iron and similar materials," this is an unpardonable error. The only difference between concrete and so-called elastic materials is a higher tension limit, and that the elastic limit and the breaking limit do not coincide in the former; they do in the latter. But in no case should the elastic limit in any material be reached, and as long as it is not reached its elastic behavior is the same in all materials.

Concrete and its preparation is to-day so well known that it would be a waste of time to go into the subject. The slightest doubt can be satisfied by reading the chapters relating to this matter in Trautwine's or in other more special treatises. That concrete must be prepared with care to develop fully its properties is not the question, because the same must be said of every building material. But with concrete the work can be superintended by one reliable man, while the actual work can be performed by inexperienced workmen.

The proportions of concrete are varied in accordance to its use as

mortar, as a water-tight shell, as merely a filling (where we sometimes use 1 to 18), and finally where it has to bear stresses.

The materials used for the last purpose are slow-setting cement, sand and gravel or broken stone. The figures 1 to 8 mean the proportion of cement to the aggregates (sand and stone together), but it is of equal importance to know the proportion of the mortar (cement and sand) to the stone, and of the cement to the sand. In both cases the voids, which should be at least filled, to produce a concrete fit for our purposes, must be considered. The proportion will consequently depend upon the size and form of the aggregates on one side and the fineness of grinding and specific weight of cement on the other.

Even in the same bridge we find, if economically built, different proportions used in different parts.

The best concrete in crown and in the outer fibers, 1 to 5 ; in the haunches, 1 to 7 ; and in spandrels, 1 to 9. These figures are the highest the writer has heard of, the usual practice being always lower, viz.: crown, 1 to 8 ; spandrels, 1 to 12.

One to 3 to 6 is recommended by Mr. A. Rella, an expert in this kind of work, wherein 1 to 3 means the mortar and 6 gravel; if broken stone is used, which for that work is the more desirable material, the proportion is changed to 1 to 3 to 5, in which the stone (5) is made up of 1 part of gravel and 4 parts of broken stone. All Monier work has been done with 1 to 3 mortar, no stone being used. Wünsch, on the other hand, sometimes used 1 to 15; but such reduction needs some experience in handling and choice of materials, and cannot be recommended generally. As the strength of concrete mainly depends on the mortar and its proportion to the stone, we specify for a mortar 1 to 3, 28 days old at least, 200 lbs. per square inch, and also the size of stone. It is, of course, quite faulty to consider neat cement tests or tests of only a few days. These structures are built for centuries, and we know that the strength of concrete is always increasing. After about 15 weeks the increase is not of material importance, but after two weeks the strength should be sufficient to carry the dead load of the bridge. On this account we sometimes see the centering removed after two weeks, sometimes after two months; but in no case should a concrete bridge be subjected to severe tests before three months, for it is clear that it will not have developed its full strength before that time.

A very striking example of this was the result of the removal of the centering of the bridge near Neuhausel. The centers of the six spans were taken down at different times. The deflection caused by the dead load was measured very accurately, and is as follows:

Span.	I.	II.	III.	IV.	V.	VI.
Time for setting in days.....	31	32	36	40	41	43
Permanent deflection, inches.....	0.59	0.67	0.55	0.47	0.24	0.08

The deflections show clearly the influence of time. Engineer Schustler recommends for this system six weeks for setting. In the loading tests made for longer periods no further permanent deflections are found.

We may now consider those properties of iron and cement which make both materials especially fit to be used together:

First.—The modulus of elasticity of concrete (after Professor Boeck) is about $\frac{1}{10}$ of that of mild steel. This great difference enables us to have in two neighboring fibers a small stress in the concrete and a 40-times larger one in the steel.

Second.—Professor Bauschinger found the cohesion between iron and concrete, after hardening, to be from 570 to 640 lbs. per square inch. This is more than the tensile strength of the best concrete. It acts like a glue if once hardened.

Third.—Concrete is the best conservator of iron. The writer knows of a case where iron rods were found perfectly rust-free having been imbedded in concrete below water level for 400 years. There are many devices designed on this important property, and for the protection of iron concrete is better than paint.

Fourth.—The thermic expansion of both materials is the same, and change of temperature will not originate secondary stresses.

Bonniceau, a French author, gives the thermic expansion of Portland cement as 0.0000143 for 1° Celsius, while iron has 0.0000145, which is practically the same.

Having now reviewed the properties of the materials used, and their application, as well as the calculation for these arches, there remains to be said something of their usefulness for highway bridges.

The main advantages of this construction are:

First.—That there is no expense for maintenance of the bridge, the iron being entirely covered.

Second.—That there is no expense necessary for maintenance of the road, or, at least, no more than for any other part of it.

Third.—That there are no vibrations and practically no noise, and that it is not affected by a change of live load.

Fourth.—That such bridges are tornado and high-water proof.

Fifth.—That they have a solid appearance, which can be architecturally developed in accordance with surroundings.

Sixth.—That their construction is cheap wherever sand and gravel are at hand.

In short, they possess all the advantages of a masonry bridge, at the cost of an iron one.

But all the first five merits have little value without the last one. In practice, prices rule the application. Can any one deny that the very best examples in bridge-building are masonry constructions? and yet how seldom do we see one.

It is certainly remarkable, therefore, that Mr. Trick, Superintendent of Bridges, Philadelphia, in building his concrete-iron bridge on Pine Road, states, like many other authors, that concrete has been for him a cheaper material than stone. On the other hand, we have the fact that the bridge near Neuhausel has been awarded to the concrete bidder, notwithstanding that the Government asked only for bids for a wooden bridge. The estimate for the wooden bridge was $12 000 for nine 40-ft. spans, and the successful bidder offered a concrete bridge of six 56-ft. spans for $13 700.

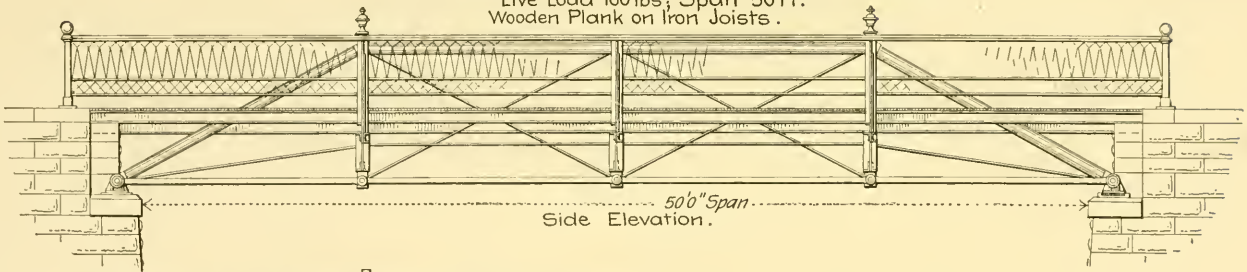
Mr. R. Wünsch states, in answer to an inquiry, that in Hungary a concrete bridge can be built as cheaply as a wooden one (with masonry pillars), and about 25% lower than an iron one.

This is, of course, not applicable to this country, because the labor here is far more expensive than there. But even if it should be possible to construct a concrete bridge for the same price as an iron bridge, it needs not a great prophet to predict a revolution in the construction of highway bridges, and the fact is that there is a general tendency to build highway bridges stronger and more independent of the live load.

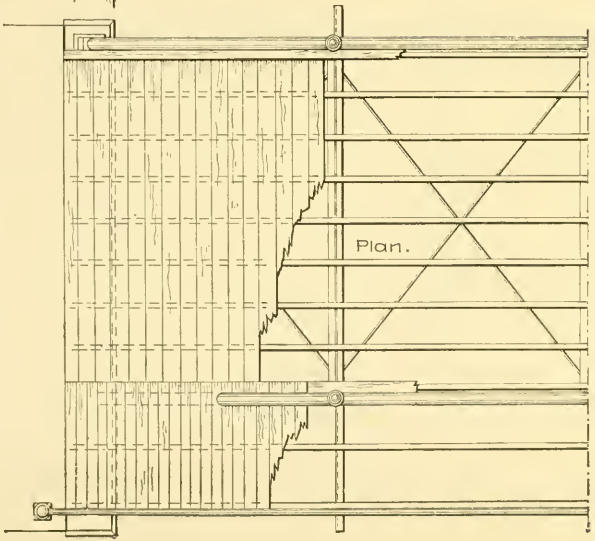
For purposes of comparison two comparative designs (Fig. 5 and Plates LX and LXI), one with 12-ft. span and heaviest loads (Fig. 5),

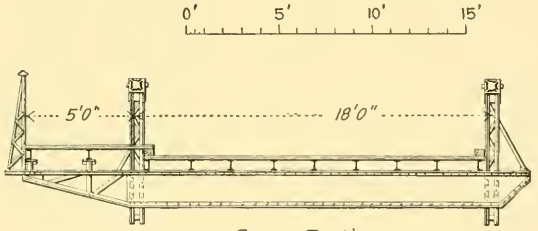
Ordinary Type of Highway Bridge (Class B.)

Live Load 100 lbs.; Span 50 Ft.
Wooden Plank on Iron Joists.



Side Elevation.





Cross Section.

Weight of Truss : 14,300 lbs.
Floor Joists : 6,950 "
21,250 lbs.

Or 18·5 lbs. of Steel per sq. ft. Road and Railing.

Fr. von Emperger.
Consulting Engineer.

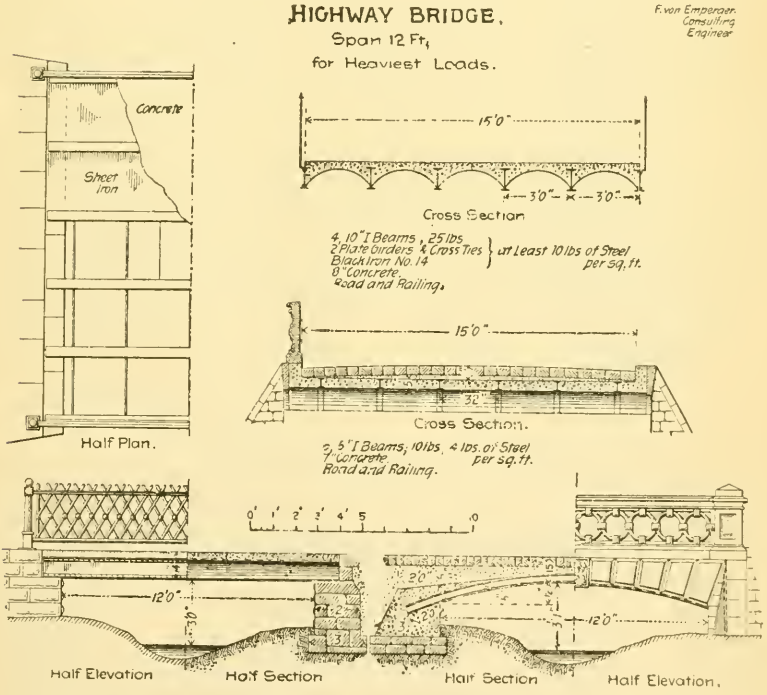
PLATE LX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 703.
VON EMPERGER ON CONCRETE-IRON BRIDGES.

comparing the Melan type with a so-called concrete bridge, which, as the writer is informed, has been very often used in this country.

The weights of material are on each of the designs. They show the old design to be three times the more expensive.

The other designs (Plates LX and LXI) show a common iron bridge, which in the writer's opinion is in price about equal to the other design of a concrete bridge. The only uncommon part is the use of iron joists. But these iron bridges are constructed for a live load of 100 lbs. and less, without regard of heavier carloads, and hardly deserve the honor of being compared with permanent structures.

FIG. 5.

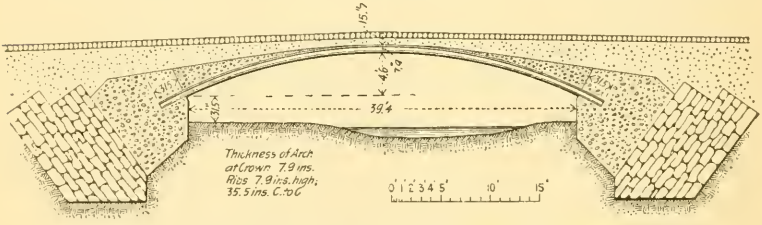


Everybody acquainted with current prices of these materials will be able to form his own judgment of the comparative cost of each type. The Melan system is the best, as it avoids any riveting in iron

and is, up to the present time, the cheapest as well as the strongest, as the tests have shown. This system is now in its second year, but the writer found it wise not to publish it before it had been thoroughly tested. Its largest field is its application to floors and vaults, to which a new system can be readily introduced, as they are not controlled by officials and depend more or less only upon the will of the owner.

It is now quite common in Europe to specify for buildings similar to office buildings, vaults with Monier arches. This is almost as common in Europe as the hollow brick floors (which are not used there at all) are here. Nearly 1 000 000 sq. ft. of the Melan type of floor have been built during the first year of its life, while of bridges the writer can only mention, besides those of $23\frac{1}{6}$ ft. span near Neustadt, two others, both of which are here reproduced, to show two different types of construction.

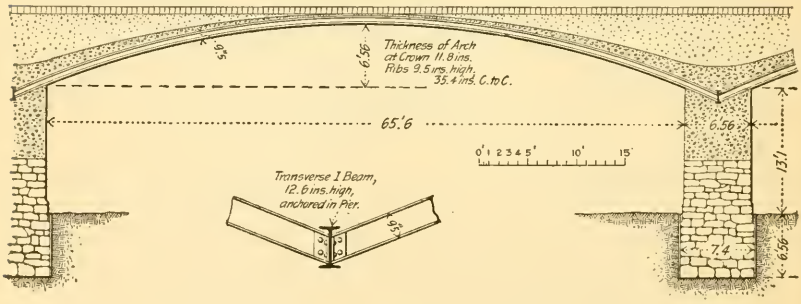
There is a bridge over the Struschka, near Oderberg (Fig. 6), with



Highway Bridge over the Struschka near Oderberg.

FIG. 6.

$39\frac{1}{4}$ ft. span, which is a type of low abutments in which beams are imbedded; and a $65\frac{1}{6}$ ft. span over the Moldau (Fig. 7) is a type with



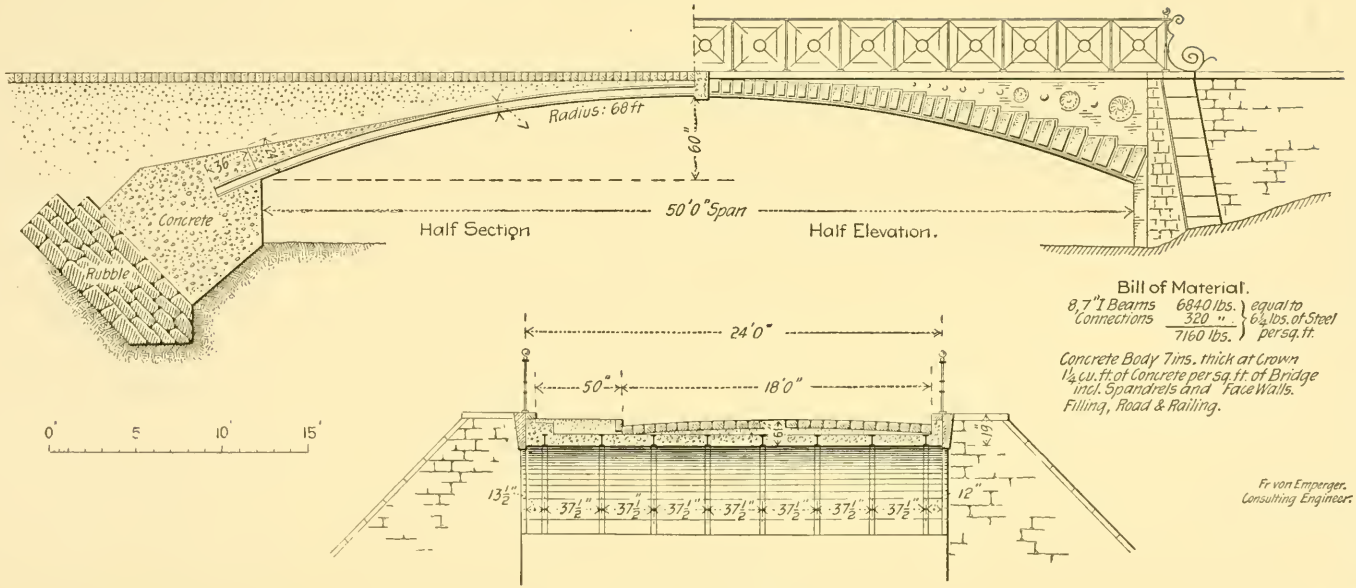
Bridge over the Moldau at Rostock, 6 Openings of 65.6 Ft.

FIG. 7.

Highway Bridge "System Melan"

Live Load 100 lbs, Span 50 Ft.

Ballast Floor



Cross Section at Apex.

Bill of Material.

8, 7" I Beams 6840 lbs.
Connections 320 "
7160 lbs.
} equal to 6¼ lbs. of Steel per sq. ft.

Concrete Body 7 ins. thick at Crown
1¼ cu. ft. of Concrete per sq. ft. of Bridge
incl. Spandrels and Face Walls.
Filling, Road & Railing.

Fr von Emperger.
Consulting Engineer.

PLATE LXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 703.
VON EMPERGER ON CONCRETE-IRON BRIDGES.

high pillars and anchorage. There is still a third type with so-called hidden abutments, but there are as yet no structures built on this plan. These bridges were built by Pittel & Brausewetter, of Vienna.

Finally, some words about exterior finish. The cheapest is the concrete finish itself, with some fancy joints. The concrete can be colored by using chips and sand from colored stones, and Mr. Ransom gives them, with the use of wooden patterns, a stonelike appearance. Although Portland cement is largely used in this country for exterior finish in harbor work and for sidewalks, there is a general opinion that it will not weather well. The use of concrete containing lime (Coignet Béton or Roman cement) is largely responsible for this opinion. In Germany they imbed large stone in the face wall. The best material, which also gives for the smallest money the richest appearance, is terra-cotta.

The average span of concrete bridges now built is 50 ft., and never has exceeded 150 ft. in Europe, to the writer's knowledge. But there is no reason why a system, which has proven better than stone, should not be extended and reach the highest span in use with iron arches, as these bridges with the use of three lead joints and built shapes (instead of the rolled ones) are entirely equal to iron ones.

To accomplish this in Europe, there is needed, not only the right man and the right place, but also a third one, the right official, who will give his permit. Those three have not yet met. The writer regards the paternal care of the Government, so far as general specifications of bridges are concerned, as a good thing; but sometimes it is also an obstacle to progress, and on this account the engineers of this country have a good opportunity to surpass European structures.

DISCUSSION.

L. L. Buck, M. Am. Soc. C. E. (by letter).—I have followed discussions in technical journals relating to the Monier system of arches with a good deal of interest, and think the paper by Mr. von Emperger should possess value. The development of the system of arches of concrete must necessarily be largely based upon empirical information coupled with sound judgment and work executed with great care.

There are many places where a durable bridge of this character could be built with economy. The author's statement, that it is

claimed for this system in flat arches that the horizontal thrust at the abutment is almost entirely done away with, should be more fully explained. If the structure acts as an arch, this cannot be entirely true. If it has sufficient thickness at the crown, it may act as a beam; or, if it has depth at the haunches and is counterweighted on the other side of the abutment, it may act as a cantilever. Temporary apparent advantages may result from combining these principles, but it appears to me that it would generally be better to depend entirely upon the arch principle and use the iron, whether in the form of wire, or of rolled shapes, to insure the integrity of the concrete under extraordinary stresses.

The statement that concrete and iron expand the same under the action of heat is to me a surprise.

Corydon T. Purdy (by letter).—I should like to call the attention of the Society to the fact that the use of steel in conjunction with concrete has already reached a considerable development in the construction of foundations for large buildings, in Chicago chiefly, and in other cities to some extent.

Mr. von Emperger's point, that these two materials are especially fit to be used together, will be appreciated by all of us who have believed this to be true and have shown our faith in that belief by our works. Ever since steel foundations, as they are generally termed, were first advocated, their enduring properties have been questioned, and even to-day some architects in that city will not use the combination. Every item of reliable evidence to show that iron or steel will not deteriorate in concrete, or *vice versa*, is therefore important in its relation to that construction.

A few words in regard to that work may not be out of place. I do not know who first conceived the idea, but it was first used by Mr. Burnham, and he well deserves the credit for its introduction. At first the metal was used only to supplement rubble masonry and concrete, then more dependence was put on the steel, and soon the masonry was omitted entirely and only enough concrete was used to give the metal a good bed and thoroughly cover and protect it. At first railroad rails were used, some of them old ones; then some beams were used, and, as soon as the iron pool was broken, beams were used almost entirely.

In rare instances riveted work has been used, and a few foundations have been made of light beams. Some thousands of tons of steel have thus been imbedded in concrete in these foundations.

Of course this is not the only solution of the foundation problem even in Chicago, and possibly not the best one, but the demand has been very pressing for something that would provide a large bearing without breaking the clay strata and yet be so thin in its vertical dimension that the basement room would not be destroyed. The steel

foundation was a cheap and natural solution of the problem. It quite completely fulfilled the requirements of that time; and if it will endure indefinitely, there are many good reasons why it may well be used for limited loads in all times.

Mr. von Emperger is very definite. He says: "Concrete is the best conservator of iron." The truth of this statement, for foundations or highway bridges or any other permanent construction for that matter, is fundamental and vitally important. If the perpetuity of iron or steel imbedded in concrete depends on conditions, it also becomes correspondingly important to have these conditions definitely fixed.

Jos. M. Wilson, M. Am. Soc. C. E. (by letter).—We have been using cement and iron or steel in combination for fire-proof floors since 1889 with great success. While not making any special claim for originality in the method employed, yet the exact arrangement adopted by us was a little different from anything that I had at that time seen elsewhere, and an illustration of it was published in one of the engineering papers in 1890.

The great desideratum in such cases is a system that shall be easy of construction without the employment of skilled mechanics and with a minimum of expense in the manufacture.

The form of construction adopted consists of the ordinary I beam girders, running from wall to wall, or from walls to columns, placed at distances apart of, say, from 10 to 18 ft., as may be required, between which are hung flat-iron or steel straps at intervals of 12 to 24 ins., the ends of each strap being bent and hooked over the top flanges of the girders, and the straps curved down, so that midway in their length they hang very close to the under surface of the intended floor. A level centering having been provided, a floor is built in of concrete, completely embedding the straps and the girders; or, if the latter are too deep for this, it is curved down on to the lower flanges of the girders. The plaster for ceiling finish is applied directly to the under surface and under the lower flanges of girders.

We have used these floors in the American Philosophical Society building, the Academy of Natural Sciences and the Drexel Institute, Philadelphia; in the Alumni building of the Rensselaer Polytechnic Institute at Troy, and are now placing them in a large training school in Indianapolis.

Those in Philadelphia were constructed of Portland cement. First, a coating on the centering of 1 in., formed of cement and sand, 1 to 2; then concrete, composed of 4 parts stone, to pass through a 2-in. ring, 2 parts sharp river sand, 1 part Portland cement; then a finishing coat on top of 1½ ins., 1 part cement to 2 parts sand, divided up into squares and tinted as may be required for the floor. If a wooden floor is desired, only 1 in. of finish is used, and nailing-strips, to which to nail the flooring boards, are bedded in light filling on top.

In the American Philosophical Society building, the spans for the first and second floors run from 8 to 16 ft.; for the former the straps being $\frac{3}{4}$ x $\frac{3}{8}$ in., 24 ins. to centers, with concrete 4 ins. thick, including finish; and for the latter the straps are $2\frac{1}{2}$ x $\frac{5}{8}$ ins., 24 ins. to centers, with the concrete 7 ins. thick. These floors were intended for 80 lbs. live load.

The third floor, of 16-ft. spans, carrying 210 lbs. loading to the square foot, being a library floor, has straps 2 x $\frac{5}{8}$ ins., 12 ins. apart, and concrete 8 ins. thick.

The Academy of Natural Sciences has five floors and roof, 17 to 18-ft. spans, all constructed in this way, the floors having 8 ins. of concrete, and straps $2\frac{1}{2}$ x $\frac{5}{8}$ ins., 18 ins. apart; live load, 100 lbs. to the square foot. The work has finished up beautifully without a sign of a crack or defect anywhere.

In the Indianapolis building there are about 67 000 sq. ft. of flooring to be laid. The concrete is to be of Louisville cement, 1 to 3 parts crushed stone with 1-in. finish on top of Dyckerhoff cement and sand, 1 to 2. The spans are about 11 ft., the straps or bars 1 x $\frac{1}{4}$ in., 12 ins. apart, and the concrete 7 ins. thick, lightened up with open cores running through between and parallel to the straps. This floor, which is to be constructed by a Cleveland firm, has been placed at so reasonable a figure as to compete successfully with wooden mill construction.

The Society may not be aware that a small concrete bridge was constructed last year over the Pennypack Creek, in Philadelphia, by the city authorities. The engineer, Mr. Carl A. Frick, states that it consists of two arches 25 ft. $4\frac{3}{4}$ ins. span each, with rise of 6 ft. 6 ins., and is built on a light skew: width out to out of parapets, 34 ft. The depth at crown is 2 ft. 3 ins.; the abutments, 7 ft. in thickness, and the pier, 6 ft. The spandrel walls, 3 ft. at the springing line, and 2 ft. at the coping. The entire work (foundations and all) has been constructed of concrete, German Portland cement, some portions being mixed of 1 part cement, 2 parts sand, 6 parts broken stone, and other portions of 1 part cement, 1 part sand, and 4 parts broken stone. Wire netting, $\frac{1}{4}$-in. wire, 12-in. mesh, was placed in each layer of the concrete.

It is stated by the engineer that a saving was effected in cost of about one-fourth over that of a stone and brick bridge. Without taking up an analytical consideration of the bridge, the depth of the arch appears to be much heavier than necessary for the span, and it is probable that with a closer approach to the vertical limits, still more saving in cost might have been effected.

Col. Wm. P. Craighill, Pres. Am. Soc. C. E.—There is one question that I would like to ask the author. On page 439 he says, "In this country, for instance, the use of concrete has been almost confined to floors." Is that intended to mean the use of concrete alone, or concrete in combinatian with iron or steel? The statement is very general.

Mr. von Emperger.—I mean concrete and iron.

Col. Craighill.—The statement is so general that I felt called upon to take exception to it, for the reason that concrete has been used for various purposes in this country for many years. For instance, for more than a half century it has been used very extensively in the fortifications of the United States. One of the earliest treatises on the subject was by General Totten, once Chief of Engineers of the Army, printed in 1838.

Foster Crowell, M. Am. Soc. C. E.—I should like to ask the author more definitely in regard to the expansion rates of iron and concrete. The reference in his paper seems to apply only to experiments upon Portland cement and iron, in which the rate is about the same. It seems to me that the expansion of a material like concrete must be as variable as the composition of the concrete. There have been quite a number of experiments made with reference to concrete as a composite material, and not only to cement. Mr. Buck has raised the question, and it seems to me that it may not be correct to assume that all concrete would expand equally with iron.

George Hill, M. Am. Soc. C. E. (by letter).—The various methods suggested for the spanning of openings by means of systems less expensive than those in use at the present time, while equally enduring and strong, have been followed with much interest by engineers. We are therefore much indebted to Mr. von Emperger for the very interesting description of the Melan system.

So far as its use for highway bridges is concerned, I refrain from expressing an opinion, since that is outside my line. We are told, however, that it is coming into extensive use for floors in Germany, and it is intimated that it would have an extensive use in that direction in this country; and it is concerning this that I wish to speak.

It is true that the actual area of floors laid in it so far is relatively small as compared with the use made of the better known styles of fire-proof floor construction in this country, but it is very much in excess of any record which we have to show for the first year after the introduction of any of our materials, and it therefore behooves us to examine the system with a great deal of care before expressing an opinion. Analyzing the arch, we find the compression is in part taken up by the concrete, and in part by the curved metal ribs; that the thrust is absorbed by the abutments, as shown by the drawings, or presumably by tie-rods in building construction, and that we further cause the concrete to act as a beam between the curved ribs, and compel it to do all of the work of which it is capable in that position. We must use the concrete in this latter way, and it is not good engineering to add to this duty an additional one by making it act as an arch, thus putting the fibers under a maximum strain in two directions. Mr. von Emperger cites tests, also, which show a bending moment at the haunches which

is apparently far beyond the strength of the concrete to resist, acting as an arch, but which is readily carried by it to the curved ribs and by them resisted. Taking this view, all the equations except 9, 10 and 11 are cleared of their complexity, and we should simply calculate the concrete as a beam between the curved ribs, establishing proper ratios at once for span and depths, and calculate the ribs for the strains which are imposed by the live and dead loads acting as arches. Under this supposition, we would have the same amount of metal required as in a beam, minus the web.

In considering the use of such a flooring we have :

First.—The use of curved ribs which require careful mill work, and, in order to secure the results which are aimed at by the author of the paper, a close union between the concrete and the iron, all the metal must be free from rust until it is used in the building.

Second.—The cement must be of uniformly high grade.

Third.—The workmanship must be honest and such as to insure a proper use of the materials.

Fourth.—In at least 50% of the cases a suspended ceiling must be provided, so as to make the bottom surfaces absolutely flat.

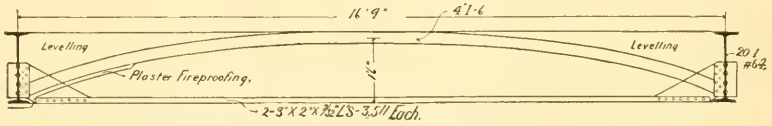


Fig. 1.

Suppose that we had an area 16 ft. 9 ins. on one side by 20 ft. on the other side to cover. By the Melan system, we would put in curved ribs, assuming 5 ft. spacing on centers, spanning the narrow way, and using the construction shown in Fig. 1, putting in tie-rods ; then we would have :

COMPARATIVE TABLE.

Melan System—

Arch ribs. Area $= \dfrac{5 \times 175 \times 16.75 \times 16.75}{8 \times 1.5 \times 12\,500} = 1.64$ sq. ins. = 4-in. I beams, 6 lbs. per foot.

Tension Member. Area the same; use two 3 x 2 x $\frac{7}{32}$-in. angles, 3.5 lbs. per foot each.

Weight per linear foot, 13 lbs. = 2.6 lbs. per square foot.

Girder. $\dfrac{21\,975 \times 82}{16\,000} = 112.5 = R$, and requires 20-in. I beams, 64 lbs. per foot.

Weight on girder, 1 280 lbs.; area supported, 335 sq. ft.; weight per square foot = 3.82 lbs.

Connections. Plates, 30.5 lbs.; angles, 21 lbs.; rivets, 10 lbs.; Total, 61.5 lbs., over an area of 83.75 sq. ft. = 0.74 lbs. per square foot.

Total metal = 2.60 + 3.82 + 0.74 = 7.16 lbs. per square foot.

Flat Arches—

Floor beams, 12-in. I beams, 40 lbs. per foot; 5 ft. 7 ins. centers,

$$\frac{20 \times 40}{20 \times 5.58} = \ldots\ldots\ldots\ldots\ldots\ldots 7.17 \text{ lbs. per square foot.}$$

Girders, 15-in. I beams, 60 lbs. per foot

$$\frac{60 \times 16.75}{335} = \ldots\ldots\ldots\ldots\ldots\ldots 3.00 \text{ " "}$$

Tie rods and connections, allow..... 1.27 " "

Total metal..... 11.44 " "

Quantities and Prices per Square Foot.

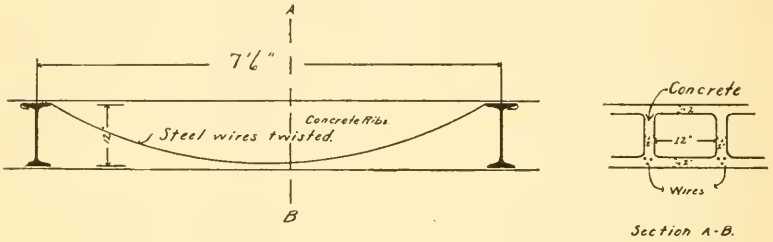
MELAN ARCH.	CENTS.	FLAT ARCH.	CENTS.
Metal, 7.16 lbs. at 35 cents.....	25.06	Metal, 11.44 lbs. at 3 cents.....	34.32
Arch concrete, 6 ins. at 35 cents.....	35.00	12-in. arch blocks.....	26.00
Leveling concrete, 4 ins. at 15 cents...	15.00	¼-in. leveling.....	5.00
Depth, 22 ins. at 2 cents.....	44.00	Depth, 16 ins. at 2 cents.....	32.00
Total.....	119.06	Total.....	97.32
Weight per square foot.....107 lbs.		Weight per square foot.....69.44 lbs.	

Flat arch saves per square foot in weight, 37.56 pounds.
" " " cost, 21.74 cents.

Clearly there is too great a difference in cost under American conditions to make the arch of general application, even where we entirely neglect the cost of the suspended ceiling. Of course there may be special cases where it would be exactly the thing, but the general case is as given above. In addition to these difficulties, there are those existing in the actual construction, which those who have devoted their entire attention to the engineering of building, know to be almost insurmountable. These are: first, with regard to the proper use of cement; second, the time which must elapse before loads can be applied; third, the lack of fireproof qualities. In engineering structures we have an inspector, thoroughly competent, constantly upon the work to see that every portion of the mortar is used under the most favorable conditions. In buildings this cannot be done, and reliance therefore, must be had, to a greater or less extent, on the knowledge of the men, who, with the most honest intentions, constantly use re-tempered mortar, mortar that has been far too highly dosed with sand,

or contains at least 50% too much water. As a consequence of these conditions, we have established the general rule that no cement should be used in tension, and that the tensile strength of the cement should not be relied on, which, if followed, would at once bar out this system.

This is said with a full knowledge of the work done by Mr. Wilson and others, and with the further knowledge that the concrete arches in use here generally only rely on the compressive strength of the concrete and are so lightly loaded under usual conditions that well-dried mud would do equally well, if given a good wearing surface, until the crucial test came. Thus assume that the section of 1 ft. is as in



Elevation Fig. 2

Fig. No. 2, we should have 64 sq. ins. in it. Assume that the weight is 60 lbs. per square foot, and that the live load is 30 lbs. per square foot, which is slightly above the average of the Boston determinations, there we would have a compression on the total area of 49.31 lbs. per square inch, which is very light. Even if we treble the live load we should only have 82.05 lbs. per square inch. This shows how great the difference is between the use of cement in the Melan system and our present use of it, and also explains why floors so built do not fail in many cases. Were we able to get thoroughly first-class concrete for floor arches we could adopt the principle described by Mr. Wilson in a more economical form by using steel wire of double the strength of the plates he uses and save probably 50% of the weight of the floor framing (see Fig. No. 2). Companies have been formed to exploit such a construction, but they have not had marked success.

We must also remember that the general use is the one to be considered, and that the building departments of the various cities in this country have had so sad an experience when trusting to the honesty of the builders that they condemn any system which is not of the simplest character and which depends on so many elements for its success.

In a country where eight months is sufficient to practically complete a building costing nearly $1 000 000, the statement that nearly three

months must elapse before the arches can be fully loaded would of itself bar them out, since often the heaviest load an arch is called on to sustain is that due to the storage of materials, and is applied before the arch is five days old and after the centering is removed.

We know that cement has been subjected to intense heat in the process of manufacture, but we also know that when heated again parts are apt to fly off, and especially where the cement is thin, as under the flanges of the beams, where it is most needed and where it would certainly fly off if touched with a stream of water while hot, thus leaving the beam unprotected against subsequent attack.

Concerning the relative strength of the two forms when used for floor construction there is no question in my mind but that the flat arch floor is the stronger under all similar conditions of treatment, since it uses mortar in compression only ; can be laid up without mortar if desired; utilizes each material in the best possible way, and in such a way that long-continued vibration cannot in any way injure it. The question of relative strength is one that we are likely to disagree upon for some time still, but, briefly summarizing the tests, we know that the fire clay alone will carry about 2 000 lbs. per square foot, in compression ; that the only tests to destruction of flat arches showed that the skewbacks were very weak, shearing off under about 1 000 lbs. per linear foot; that properly designed skewbacks have carried as high as 5 000 lbs. per linear foot without failure. That in the ordinary flat arch there is an area transmitting compression of about 30 sq. ins., and a joint area of about 144 sq. ins. in the comparative case given, and therefore we should have a safe uniform load on this 12-in. arch of (safety factor, 10) :

$$L = \frac{W}{5.58} = \frac{8 \times 30 \times 8 \times 200}{67 \times 5.58} = \frac{384\,000}{373.86} = 1\,027.$$

This is on the assumption that the whole area of the arch acts ; if only three-quarters acts, then the load becomes 770 lbs. per square foot. This uses only well-known formulas, and a conservative value of the crushing strength. With the Melan system, we must assume the extreme fiber strain at 90 lbs. per square inch, in order to obtain a reasonable thickness of slab between arch members and this is confessedly very high (see page 449).

We cannot determine from the test given what the concrete of the Melan arch would carry, since failure was occasioned by a load concentrated on the metal rib and concrete at one point, and when the rib bent the cement ruptured in every direction (see Plate LIX). The author states, page 453, that 640 lbs. gives a factor of 7.1. Whereas, with the flat arch, according to our figuring, the factor is 44. There is no doubt but that we need an exhaustive series of tests to determine what the strength of our flat-arch floors really amounts to.

Concerning the reference by Mr. Purdy to the use of cements and steel in Chicago, I would refer you to the *Engineering News*, pages 116, 265, 312, 415, Vol. XXVI, which quite clearly demonstrates to me that this use was not good engineering. Both then and now, it would cost more than a combination of girders designed according to the generally accepted theories of flexure.

Joseph M. Wilson, Vice-President Am. Soc. C. E.—I think I stated clearly that I made no special claim to great originality in my form of construction. I am aware that construction of a general character similar to what we have adopted has been used in San Francisco and perhaps in other places, the detailed arrangement in our case being different. Some of the floors we have built have been finished now for four or five years.

It is possible that the building laws of the city of New York may prevent the use of floors constructed as these are, but I do not think that one of the reasons mentioned, to wit, the question of the builder not doing his work properly, should be an excuse for its rejection.

So far as plastering is concerned, the ceilings were plastered directly on the cement, and there has been no staining or other trouble with them. The ceilings in the extension to the Academy of Natural Sciences in Philadelphia could not look better. They can be considered a perfect piece of work, although they have been exposed all winter without heat in the building, as no boilers have yet been provided, and the building has been up since last fall. The work could not be in better condition than these ceilings are.

I think also the question of expansion and contraction is somewhat of a bugbear. In buildings the conditions are a little different from those of outside work. The variations of temperature are never very great, as the walls protect the interior from cold in winter and heat in summer, and in winter there is always artificial heat when the buildings are occupied. I do not think the extremes are very great in any of our buildings.

W. G. Triest, Jun. Am. Soc. C. E.—The span of 150 ft. has been mentioned as the maximum which has been used for concrete bridges. By concrete bridges we must understand iron-concrete bridges here, but it will not be inopportune to call attention to a concrete-arch bridge of 164-ft. span, which has recently been built over the Danube at Munderkingen, Wurtemberg. This bridge is the more interesting, as, in a sense, it is a combination bridge, also having some iron members which have mostly been used in iron construction only. The rise of the arch is 1 in 10, and its width, 24 ft. 3 ins. No spandril filling was used in order to keep down the dead load. At the apex and at the skewbacks cast-iron half cylinder and socket joints were inserted, in order to render harmless the effects of the settling. The arch settled $2\frac{3}{4}$ ins., when the centers were struck, and $1\frac{5}{8}$ ins.

more when the roadbed was put on. The skewback joints were then covered with concrete, while the apex joint remained free to allow of a movement due to changes of temperature.

An instructive proof of the preserving power of concrete was furnished last year. The cast-iron concrete-filled pillars of a viaduct had been taken down. When they were broken up, a wrench was found inside of the concrete, having been buried therein for 22 years. It had kept a metallic blank surface under the concrete, while part of the wrench that had been imbedded in coal ashes had suffered greatly by rust.

Edgar B. Gosling, Jun. Am. Soc. C. E.—I have made large concrete blocks which, after long life, show perfectly smooth and good surfaces, and I therefore do not agree with Mr. O'Rourke that the cause of peeling or shelling off at the surface is due to differences of expansion and contraction in the interior, and on the surface of the stone. To further bear out this point, I would state that I have had considerable practice in the manufacture of artificial stone, having, among other work, put up a structure about 180 x 200 ft., entirely walls, etc., of Béton Coignet. This structure had columns 15½ ft. in height by 18 ins. in diameter, in one piece, and I have found that the surfaces are perfect, and that expansion and contraction have not the effect mentioned on well-made and properly rammed concrete or béton. I should like to take exception to the remark made by Mr. von Emperger, on page 457 of his paper, in which he states that "Coignet Béton is largely responsible for the general opinion that concrete will not weather well"; as this statement (although very handy to cover the many drawbacks of the Melan system) is by no means borne out by facts.

Carl Gayler, M. Am. Soc. C. E. (by letter).—The use of Portland cement concrete has wrought a revolution in all branches of civil engineering, and it seems that we are only in the beginning of the radical changes, which in bridge work, sewers, water works, railroads, etc., are following its introduction. Any new form, therefore, under which this material can be utilized is worthy of the closest attention, and this is what gives the paper under discussion an unusual value.

The shallow and exceedingly thin concrete arches which have of late years been built in Germany and Austria have excited a good deal of surprise on this side of the Atlantic. They are, undoubtedly, the most convincing proofs of the wonderful qualities of the Portland cement, of which the Germans, judging from papers read in Chicago, are justly proud. Considered as advertisements of German Portland cement alone, they seem to be worth the money which they cost. They have, so far, not been imitated in this country, although we cannot be accused of being slow in appreciating and appropriating for our own use any new valuable features in engineering from other parts of the

world, and it is very likely that, whenever such arches come into general use, their proportions will be considerably modified; to rely on concrete to resist tensile strains when it is obvious that the unavoidable settlement, after removal of the centers, may start a crack at any place, appears like unsound engineering. With all due respect for the minds who planned and executed these bold arches, the question may well be asked whether an engineer is justified in reducing the dimensions of a bridge to such evidently dangerous limits; as, for instance, 128-ft., and 150-ft. spans with a thickness of crown of $6\frac{2}{3}$ ins., any more than the bridge engineer, who, in designing an iron bridge, strains the metal close to the elastic limit. It looks as if the whole talent and ingenuity of these engineers had been employed on the problem of building the thinnest possible arches with a material estimated in value as one of the precious metals.

The fact that steel nettings and steel frames of various designs, imbedded in the concrete, are now being used for such arches, seems to prove that the European engineers themselves are not without their misgivings as to the safety of these structures, and the author of the paper discusses three different methods now in use in Germany, Austro-Hungary and Switzerland.

The oldest (comparatively) of these, the so-called Monier system, has been known for some time. It will probably never be used for spans exceeding 50 ft., as it admits in nowise of a calculation of its strength, the engineer having to be guided entirely by the results of tests made on test arches. The second system, introduced by Mr. Wünsch, of Buda-Pesth, is a great improvement over the former, as it admits of a more intelligent proportioning of the metal work. It is not stated quite clearly in the paper whether the iron frames are designed so as to resist the loads by themselves, the concrete only acting as spandril and lateral bracing, but we can infer this from Mr. von Emperger's remark after speaking of the failure of one of the Monier arches, "that this difficulty, *i. e.*, unsafety of the arch owing to the use of worthless concrete, has been overcome by the other systems now in use." The Wünsch system is also superior to the last of the three systems (Melan) because the tensile strains of both the upper and under sides of the arch are resisted by metal.

For the system of Professor Melan, to which Mr. von Emperger gives the preference, the claim is made, supported by a calculation, that the concrete and the beams in the arch work together. The full strength of the concrete, in addition to the strength of the steel beams, is counted upon as acting as an arch, and at the same time as a very efficient lateral and vertical bracing. The claim is made, for the first time in the history of civil engineering, that iron and stone, in spite of their different strength, elasticity and thermic expansion, if properly proportioned, act together like one material. The case is entirely

different from the use of anchor bolts in a bridge pier, because there the metal is held only at the ends and is stretched and released without account being taken of the contact with the masonry; it is also different from the use of steel beams with concrete arches between them, because the steel beams and the arches each carry their own share of the loads individually, but we would have a case similar to what is claimed for the Melan system if we would design a column or pillar of steel and concrete assuming the strength of the pillar to be equal to the sum of the strength of the steel and concrete.

In regard to those properties of iron and cement which, in Mr. von Emperger's opinion, make both materials "especially fit to work together," I wish to say:

1. We know that the modulus of elasticity of different kinds of steel shows little variation, but we also know, and we can see from the paper under discussion, that the modulus of elasticity of Portland cement concrete is a quantity about which we know very little, the 750 000 assumed in the calculation of Mr. Melan is about one-half of the figure we have been in the habit of assuming, and is obtained from experiments which have not been published yet.

3. I agree with the author that concrete is the best preserver of iron, but the fact that the underside of the steel beams is not protected is objectionable in a system of highway bridges for which no expense of maintenance is claimed.

4. The remarkable fact, given on the authority of Mr. Bonniceau, that the thermic expansion of Portland cement (should this read Portland cement concrete ?) is within $\frac{2}{10\,000\,000}$ parts the same for both materials for 1° Celsius may, for want of sufficient data, go unchallenged, but surely, steel being a better conductor of heat than concrete, there must be considerable difference, at the same moment, in the expansion of the two materials. The influence of temperature would, in view of the exceedingly rapid changes of temperature in this country be far greater here than in European countries.

I am willing to acknowledge that for short spans, and in cases where a graceful appearance of the structure is desirable, the Melan system has its advantages; but I believe that the European official with whom Mr. von Emperger finds fault for once acts wisely in limiting the lengths of the spans.

Bernt Berger, Assoc. M. Am. Soc. C. E.—The description of the different systems of concrete iron construction in Mr. von Emperger's paper is very interesting, and the tests enumerated would apparently point to the possibility of wide application of the systems to bridge construction.

There are, however, some points in the paper which have not been made quite clear, and in other places the author has made more sweep-

ing claims for this kind of construction than seem warranted by an unbiased study of the tests described.

The structures are treated as arches. In the description of the Wünsch system arch it is stated that no horizontal motion of the abutments was measurable, and that the horizontal thrust was changed to a lift. No motion may have been measurable, but that the thrust must have been there is readily judged from an inspection of Fig. 3 in the text. Evidently it acted more as a plain beam and a cantilever combined than as an arch. In a cantilever we have, however, a horizontal thrust to take care of, and in this case the abutments were strong enough to resist it. How columns would have acted if the "arch" had been put on top of them, as suggested, is a different thing.

The author's claims of superiority, for the Melan system especially, suffer somewhat by such statements, as "it is not difficult to show that these bent beams would safely carry alone the load intended," and, further down, in comparing the three systems described: "It must be remembered, first, that any of them will do, because both materials will do it as well alone."

We have as yet no actual proof that concrete and iron will act so well together as the author claims. His description of the tests made in Austria does not prove that they act so well together in actual service, and for any length of time. The application of the combination, for instance, to building foundations, especially in Chicago, has been followed with a great deal of interest, and doubt as well, and developments, in one direction or another, will be eagerly watched; but the method is of so recent origin that no reliable conclusions can as yet be drawn. The use on the Pacific coast of beams and girders of concrete, with iron rods imbedded, to take tensile strains, seems to have met with some degree of success. It would be interesting in this connection, to hear of observations made where such girders have been used for sidewalk vaults, as I understand they have, and exposed to variable loading. This form of the combination has, however, the advantage of having the iron rods so placed as to make the strains which come upon them more clearly defined, and no reversal of strains takes place. It was also found advantageous to use twisted rods in order to give the concrete a better grip upon them.

The tests of concrete-iron arches, described by Mr. von Emperger, certainly show favorable results. But it must be remembered that these arches were built for the express purpose of being tested. They had been given a certain time, varying in the different cases, to settle and harden, but none of them had been in actual use to carry traffic. The author claims for this system of construction freedom from vibrations, a rather bold claim, especially for arches of so slender dimensions. A question is, whether this kind of an arch would not, after having been exposed to the traffic intended for it for some years, have

suffered so much from just such vibrations as to seriously impair that glue-like bond.

Another and even more serious source of danger to that bond would be the effect of changes of temperature. Even if Portland cement has the same thermic expansion as iron, that by no means proves that concrete has. A mass, consisting to such a large extent of stone as concrete, would rather be supposed to behave like stone, and we know that stone and iron are not alike in this respect. Besides, the thermic conductivity of iron is much greater than that of concrete. Under changes of temperature the iron will be heated or cooled much quicker, and to a far greater extent, with a corresponding greater expansion or contraction, than the neighboring concrete, if not imbedded sufficiently deep, and the illustrations in the paper show the iron ribs practically exposed. That the result would be disastrous to the bond seems clear.

I think, therefore, that the Melan system cannot be considered thoroughly tested before more light has been thrown on these points. If a test could be made of an arch, which had been in use for some years, it might give us some valuable information in this respect, but until such information can be had, it would seem only proper to move cautiously in adopting the proposed system.

There appears to be no reason why concrete filling cannot be used between the iron or steel ribs of an arch instead of the ordinary floor construction, if it is thought desirable, and it would, besides, do useful service as a continuous stiffener of the ribs. Whether it would be economical to use concrete in this way is another question. It would, however, seem advisable to introduce iron rods between the ribs, to prevent them from spreading, and not trust to the rather uncertain adhesion between the two materials.

The advantage due to steel having a modulus of elasticity about 40 times higher than that of concrete is not so manifest. Limiting the tensile strain on concrete to 42 lbs. per square inch, as introduced by Mr. von Emperger in the calculations, the tensile strain on mild steel should not exceed $40 \times 42 = 1\,680$ lbs. per square inch, if the elastic behavior of the two materials should be alike.

The tensile strength of concrete we know comparatively little about. The author makes the statement, that it can be taken from 290 lbs. to 340 lbs. per square inch, and infers that these figures may be doubled in determining the transverse strength of concrete beams. These figures seem, however, very high. General Gillmore, in his "Practical Treatise on Limes and Hydraulic Cements," gives the results of experiments made on the transverse strength of concrete beams at Boulogne-sur-Mer, in 1858. The *Engineering News*, of May 17th, 1890, reprinted from an Austrian technical journal the results of a series of transverse tests of concrete beams, made at Ymuiden, Holland, in 1888–89.

In both these places, the tensile strength of concrete, determined by transverse tests, is found very much below the figures given by Mr. von Emperger.

In the calculation of concrete-iron arches given in the paper, no attention is paid to temperature strains, which, in fixed arches of so little rise as those shown, may add considerably to the strains due to dead and live loads, while the difference in thermic conductivity will further induce secondary stresses.

Everything considered, it would seem that we ought to be very conservative in proportioning concrete sections, for tensile strains, especially in combination with iron or steel.

In this connection, it should also be borne in mind that in proposing new forms of construction, a fair comparison with present forms calls for equally safe strains in the structures to be compared. Claims of economy of material should not be based on the introduction of higher unit strains. Such economy can be gained as well in our present forms of construction.

Finally, it has not been made clear how an arch on the Melan system can be made more tornado and high-water proof than any present form of well-designed bridge.

W. R. Hutton, M. Am. Soc. C. E. (by letter).—Some of the statements on page 440 of this paper are inaccurate, and should not go into the *Transactions* of our Society without correction, although they do not directly affect the matter under discussion.

The arch over the nave of St. Peter's is a semicircle (not a parabola) of 90-ft. span. The top of the imposts is 110 ft. above the pavement, making the total height under the key 155 ft.

Pont d'Alma is not a railway viaduct, but a very handsome street-bridge in Paris. The rise of its oval arches is one-fifth of their span. It is not built of concrete, but of dressed stone backed with rubble masonry. Pont Napoléon, which carries a city street as well as a railway, also within the limits of Paris, is of similar, though less luxurious, construction. Notes of the German bridges referred to are not at hand.

If the concrete arch of 150-ft. span was made with a hinge at the crown, the depth of $6\frac{3}{4}$ ins. at that point is not extraordinary. If otherwise, some details of its construction would be of interest.

The Wünsch system described in the paper is the same in principle as the Cadiat system for iron arches at one time much in vogue on the Continent of Europe. One who has studied the system remarks concerning it, that the top chord or stringer is liable to excessive temperature strains which may easily exceed the elastic limit of the material. This actually occurred at the Pont d'Arcole in Paris, in which there is a perceptible dislocation at the center, at which point the rib is but 16 ins. deep. These strains would be relieved, and others not

increased by cutting it in two at the center. A second defect of the system is the racking of the piers by the alternate thrusts and pulls due to local loads and changes of temperature. The abutments of the bridge of Szegedin are said to be dilapidated from this cause.

The use of iron in combination with masonry to supply the want of tensile strength in the latter is not new. Indeed, one-half of a Wünsch "arch" of 120-ft. span and about 11-ft. rise, cut in two at the middle, would represent Brunel's model half-arch, built of brick and hoop iron about 50 years ago.

As to the so-called Monier system the principle is correct, to build into the concrete iron bars or straps at points where tensile stresses come, with the condition that the metal shall be used in such form and manner that its value can be ascertained. The concrete floor described by Mr. Wilson fulfills this condition.

The Melan system appears to be an entirely different thing. In it no attempt is made to supply directly with metal the deficiency in tensile strength of concrete. It is a combination in one structure of materials entirely distinct in their characteristics, in which combination the moment of inertia of the sections is inversely proportioned to the modulus of elasticity of each material.

It is difficult to realize that such combinations would result in sections so much more slender than could be adopted with either material used alone.

The theoretical investigation of the examples given would involve a labor which cannot now be devoted to the subject.

It may, however, be concluded that in the hands of one familiar with the theory of the elastic arch, and thoroughly acquainted with the materials used, some economies may be realized—economies which in unskillful hands may be too dearly bought.

George A. Just, M. Am. Soc. C. E.—In reply to the inquiry of Mr. Crowell, it may be stated that Mr. Hyatt in his book (which the Chairman announces Mr. Worthen has sent us to-night) proves the extension of both materials to be equal, in iron-concrete beams, whether under load or fire, at least to his own satisfaction, and concludes that, "in combining concrete with iron for building purposes, the two materials may be regarded as practically homogeneous."

The application to structures, however, of iron in various shapes in combination with concrete, must very much antedate the period fixed by the author of the paper. To trace the origin of this device would be much like tracing back the origin of the telephone or the first use of the cantilever principle in bridge work.

In fact the book referred to by our Chairman was published in London in 1877, for private circulation, by Thaddeus Hyatt, with the rather lengthy title: "An Account of Some Experiments with Portland Cement Concrete, Combined with Iron, as a Building Material

with Reference to Economy of Metal in Construction, and for Security against Fire in the Making of Roofs, Floors and Walking Surfaces."

Mr. Hyatt first quotes from Mr. Fairbairn's work, "On the Application of Cast and Wrought Iron to Building Purposes," and shows that even at that time two systems had been introduced in France styled "Système Vaux" and "Système Thuasné." In the former the iron of the combination consisted of wrought plates bound together by tie rods, which are crossed by other rods supporting the ceiling. In the latter system rolled iron joists were used instead of plates. Mr. Hyatt also mentions his factory building in Farringdon Road, London, as being constructed on similar lines, and to win favor for his method had about fifty of such combination beams tested to destruction by Mr. Kirkaldy. Mr. Hyatt also prints a letter addressed to him by that eminent American experimenter, R. G. Hatfield, showing that the former made and had tested combination beams as early as 1855.

From his investigations he concludes that the method of combining iron and steel with concrete, "should be applicable also to bridge construction," and urges its use in the constru tion of chimneys, lighthouses, stairways, etc.

It may be added that in 1883 there was published in the papers of the American Society of Mechanical Engineers an article by Mr. W. E. Ward, entitled "Béton in Combination with Iron as a Building Material," wherein he describes the construction of a dwelling at Port Chester, N. Y., where "not only the external and internal walls, cornices and towers," were constructed of béton, "but all of the beams, floors and roofs were exclusively made of béton, re-enforced with light iron beams and rods," built by him in 1875. So much for the history or antiquity of this method of construction.

Since the author of the paper states that this system finds, "its largest field in its application to floors and vaults," it is perhaps permissible to touch upon its use in such constructions. He is, however, in error when he states that with us its introduction is "not controlled by officials," and that the paternal care of government is entirely lacking, requiring no special permit or supervision.

In illustration: The law governing building operations in New York City says all arches placed between iron or steel floor beams shall be of brick or stone, or "sectional hollow brick of hard burnt clay, porous terra-cotta, or some equally good fire-proof material." Under the latter clause a special application must be made for the use of plain or combined concrete.

Such applications have heretofore generally been denied by those in authority, because the safety of such floors depends so much on the honesty of the contractor, the care of the mechanic, the action of frost, and the not too early removal of the centering.

When it is borne in mind that such official inspections are, after all, often only superficial, not to say dishonest, and that the author himself records disasters by dishonest work, or too early removal of centers, then the grounds of refusal given do not seem entirely faulty or unjust.

There now exist in this market companies prepared to execute work of this composite character, and some of the methods employed, notably those in which single wires are passed continuously over the upper flanges of the beams, and depressed between the bays to within 1 in. or so of the ceiling line by lengths of gas pipes, the whole being filled in with cement concrete, imbedding both beams and wires, have advantages not possessed by any of the systems set forth in the paper and would no doubt prove keen competitors with the same, both from technical and economic points of view.

In making a comparison of bridges built in this manner with those built entirely of iron or steel, it would seem that aside from the questions of relative cost and strength, the former should suffer, if, as the author states, they will not have developed their full strength "before three months," for generally bridges are not built until there is an immediate demand for their use.

When the author speaks of a system that changes a "horizontal thrust to a lift," or when he speaks of "concrete as inelastic, like cast iron," he is not altogether clear. Cast iron is certainly elastic, even if more so than concrete, and the elimination of thrust in an arch means a violation of the accepted principles of statics.

J. F. O'Rourke, M. Am. Soc. C. E.—It seems to me, from reading this paper, that the writer of it advocates the use of concrete arches with iron beams. There have been several objections made to this to-night; that there is an uncertainty as to what the character of the concrete might be, whether it would be good concrete, or whether the contractor would be honest. I claim that if there is any building material in general employment in this country it is concrete, and it is coming into use more and more all the time, so we ought not to consider so much whether you can get good concrete as whether good concrete would answer. The reason why we have not adopted concrete masonry without stone facing in this part of the country, where we have cold weather, is that the concrete does not hold together on the surface. The reason is very plain. The expansion and contraction of concrete is about the same as iron, but it only heats on the surface; the result is that you have a shell that is expanding and contracting, with daily variations of temperature, while the interior of it maintains its temperature practically unchanged except from season to season. I cannot recollect a single instance of concrete *en masse* around New York in which that effect has not appeared. Sound it when new, and it is like stone. In the course of time, the surface be-

comes loose and shells off. Another layer loosens and falls off in the same way, and thus the process of disintegration goes on. Generally speaking, an arch made of concrete will eventually disintegrate in this climate. When you put it in conjunction with iron, you introduce another element of disintegration. Any engineer who has used heavy masonry and heavy iron construction upon it knows that the masonry expands and contracts to a hardly appreciable extent in comparison with the metal. It is plain that if you put an iron beam in concrete, with the flanges flush with the surface, that the beam will not be of the same temperature as the concrete, when the changes in temperature are taking place rapidly. We know that the iron will creep in and out of the concrete. The fact that such structures are not generally adopted in this country is because this is fully understood.

I think, also, that there is a great deal to be said on the subject of concrete. There are wide differences of opinion as to how it should be mixed. The author says it ought to be rammed in in 6-in. layers. To do this it must be more or less dry. Some years ago I had occasion, for a certain bridge in the South, to build all concrete piers, and when the boxing was removed from the first of them, the surface was rough; on the surface, for the remaining piers, the concrete was mixed so wet that it puddled itself, gave smooth surfaces, and was as dense as if rammed. It may be stated, as a general principle, that concrete takes up the amount of water necessary, rejecting the surplus before setting.

This matter of concrete, I think, has not been sufficiently discussed among engineers, and it is very desirable to have any paper that brings out a discussion on the subject.

It is fair enough to say, in conclusion, that I do not believe that there is any likelihood that the general practice will be departed from and that we shall have concrete built here without stone facing above the ground, excepting blocks whose sizes permit of this surface motion within the elastic limit.

John Bogart, M. Am. Soc. C. E.—There are some excellent examples of concrete construction in some of the hydraulic concrete pavements which have been laid in New York and in many other places. Those which have been properly made have continued in good condition for many years under circumstances of extreme exposure and constant wear. The pavements of Broadway and of Chambers Street, adjacent to the City Hall Park, of New York, were laid twenty years ago. They were laid under carefully prepared specifications, and they are now in good condition notwithstanding the enormous travel of pedestrians which has been constantly upon them. There are other pavements which seemed, when laid, to be of similar construction, but which failed after short service. The difference was in the materials used and in their proportions and manipulation.

Bernard R. Green, M. Am. Soc. C. E. (by letter).—Mr. von Emperger's collection of examples of thin concrete arches reinforced with wrought iron in some of its many new and specially adapted forms is a valuable contribution. It will increase the confidence of engineers in the utility, safety and economy of cement mortar and concrete when intelligently and skilfully handled. Only those of wide experience, however, in the personal testing and manipulation of cement, sands and ballast of several kinds and qualities, are likely to fully appreciate the real nature of concrete. It is so largely a matter of experience and skilful, honest manipulation that the successful attainment of higher results has not infrequently been the reward of the workman rather than of the theorist, as in the case of Jean Monier. Such manipulation will always give satisfactory results even with very ordinary materials, while the reverse, with even the best of cements, leads to disappointment. For the construction of good concrete work, therefore, experience is the chief teacher—the art cannot be learned from books.

Experienced makers of hydraulic cement concrete will not be surprised at the strength and stiffness of the slender arches described in the paper. The immense strength of the egg-shell and of a continuous plate of any material, whether flat, bent or dished, indicates the possibilities of these forms when carefully made in cement mortar or concrete and thoroughly hardened. It is not at all easy, however, to get every one of a gang of workmen to follow out all day long every detail of the mixing, placing and protecting of fine concrete work. Some one or more will slight it somewhere at some time, and whenever this happens a weak spot occurs in the structure. The more successful the discipline, the more homogeneous and economical will be the product, and more uniform the strength. Thus, with constant vigilance, safe arches, domes, sewers and pavements of the slenderest forms and least quantities of material may be made. Whenever suitable machinery can be applied, uniform results are always assured, because it never becomes tired or careless.

The thinnest of arch rings, with spandrels filled, and the crown more or less covered, with even loose material, will carry great local loads through the distributing effect of the filling. The better the filling, the stiffer the arch, and so on. With strong cement mortar or fine concrete, containing wire or properly disposed light iron of some of the many new and extremely light sections now so readily and cheaply obtained in very long continuous lengths, selected and contrived intelligently for each new case of plate, arch or dome, almost wonderful results may be easily secured. Where we know what can be accomplished without the iron reinforcement, we may anticipate something very superior with it. In no case, however, may we abuse the cement at any point, from the time it is wetted until after the

expiration of the few weeks necessary for sound induration, without seriously impairing the structure. We must also look to the abutments and supports.

Hydraulic cement, fairly treated, is the engineer's friend, and, even like a faithful dog, not wholly alienated by abuse. The scope of its usefulness is still rapidly widening. In light brick and terra-cotta arching, as the matrix, it serves a most valuable purpose by converting the arch into a monolith like the continuous concrete arches described in the paper.

The so-called "Gustavino" arch, recently introduced in this country, is nothing more than the careful and intelligent construction of arch and dome shells with small rectangular 1-in. plates of hard terra-cotta, laid together on the flat-like tiles in either square or herring-bone bond. Two, three or four courses of these tiles are overlaid like the onion, breaking joints, with good Portland cement mortar in the joints and between the layers or concentric shells. When hardened the result is an extremely tough and stiff shell, capable of resisting very considerable local loads or shocks. A dome of this sort is like an egg shell on a large scale. Such construction, very useful and economical for many purposes, only illustrates what may be done with common bricks and cement mortar properly used. As a contribution to the published test of such arches, as well as of hollow terra-cotta work, much in the same line, I give the following, shown in Figs. 1 and 2, page 479.

Both arches were of hard red brick and Maryland cement mortar, 1 to 2, laid up in the regular course of construction without reference to special testing. "Cumberland" cement was used in the former, which was six months old at the time of the test, and "Round Top" cement in the latter, which was two years old.

The first arch (Fig. 1) was 34 ft. 5¾ ins. long, 10 ft. 5½ ins. span, and 11½ ins. rise. The haunches were one brick and the middle one-half brick thick, as shown, and the abutments were very heavy unyielding walls. One end of the arch was loaded close to the head, which was free, with a column of dry bricks, over the half-brick section of the arch ring, on an area of 7 ft. 6 ins. along the crown by 7 ft. transversely. About three-quarters of the load was applied during the first two days, and the remainder on the sixth day, when the test was stopped through fear that the column, then 13 ft. high, would topple over. The total load was 56 450 lbs., or 1 075 lbs. per square foot of loaded area. The deflection was $\frac{5}{16}$ in., at the crown, which entirely disappeared on removal of the load, and there was no crack or other sign of weakness.

The second arch (Fig. 2) was groined, springing from very heavy and unyielding abutments, between which the four heads were tightly enclosed, but not bonded thereto more than 3 ft. from the groin on the

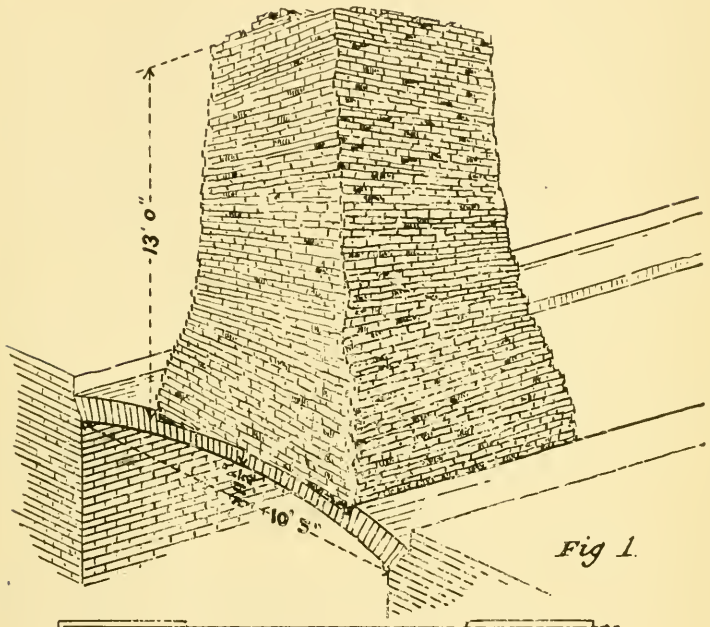


Fig 1.

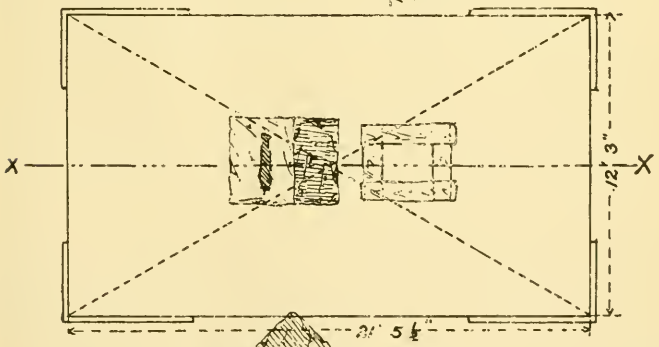
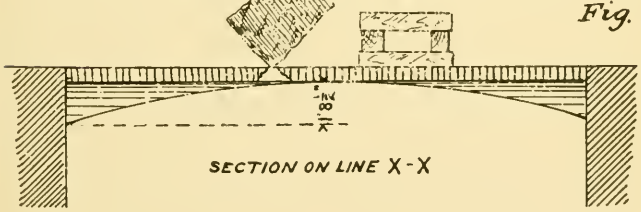


Fig. 2.



narrow head and 5 ft. on the wide head. These groined sections were 13 ins. thick, and all the remainder of the arch 8 ins., or one brick thick. The test of this arch consisted in tilting over on to the middle of its crown a nearly cubical block of granite, weighing 2¼ tons, from a blocking 2 ft. 3 ins. above the arch. The stone struck partially edge and corner wise, sending a heavy vibrating shock throughout the adjoining series of floor arches, of which this was one of five of about the same size, besides several other smaller arches. The sound and effect were similar to those of a heavy blow on a bass drum, and the only fracture or disturbance of any sort was a puncture of about 24 x 8 ins. splaying out below. The brick work spawled away like a monolith, quite regardless of joints or bond.

Fr. von Emperger, M. Austrian Soc. of Engrs. and Archts. (by letter).—I desire first to eliminate from this discussion such statements as cannot be found in my paper.

Messrs. Buck, Berger and Just claim that I have said that there will be no horizontal thrust in an arch. This is based on a statement regarding the Wünsch arch, page 444, beginning, "the inventor claims," etc., and which does not give my own opinion in this question. I avoided doing so, because I do not believe that it would have any value without further testing.

Mr. Wünsch's claims, however, may be true, inasmuch as a concrete beam, so shaped, may not act as an arch at all (see Plate LVII). I may further refer to an iron bridge built on this principle, the Stephanie bridge in Vienna. It is a cantilever bridge, with plate girders of an arch-like shape, and I understand that Mr. Buck and Mr. Berger refer to this point of view.

Mr. Just states that I have called "concrete inelastic like cast iron." On page 451, he will find that I quoted a technical paper as doing so, and stated that it is an unpardonable error.

Mr. Just quotes me as saying, "They will not have developed their full strength before three months."

This is an error in the same direction. The quotation is so far right, but not in the meaning given by this gentleman, that a concrete bridge could not be used before that time. The bridge will, after that time, be ten times as safe, and after three years, for instance, even more so. But this later increase will not be remarkable, and, therefore, the strength, after three months, may be taken as the standard. Every concrete bridge can be used immediately after completion, and centers will be struck as soon as it can be used without them, and the time for taking down centers varies from two months to two weeks (see page 452). Centers for floors are struck after one week, and I know of cases where this has been done after four days without bad results.

The time of striking centers must be determinated by the span, by the stress the dead load produces, by the kind of live load (railroad

bridges require the longest time), and by the extent to which the iron is able to carry the loads alone; as, for instance, in floors. For this reason the necessary time for setting will be longer with the Monier and Wünsch and shorter with the Melan system. The tremor from traffic has influence only during setting, and tests of cubes of concrete manufactured on a moving engine show a reduction of only 10% in strength, as compared with similar cubes which had set quietly. Mr. Just quotes me as saying that floors for buildings can be laid in this country without permits. On the last page of the paper, he will find that this had reference to highway bridges only.

So far as floors are concerned, all the building laws of the world, where there are any, have the same requirement as the New York law. Permits must be had for special or new constructions. But I hardly think that this can be taken as equal in value to a scientific testimonial like the permit for a bridge in Europe. It is only the opinion of an official who may not be competent, and depends a greal deal upon the personal influence of the promoter, or on other accessories. The statement given on page 456 applies only to Europe, and does not refer to this country.

In answer to Mr. Crowell, I may say that some recent tests of mortar (Meier-Malstatt), 1 to 6, did not show any remarkable change in expansion. Further, I would like to mention the tests of Dr. Früling, showing that the tensile strength of concrete is not affected by heat below red heat. This latter point is very important, but seems to me not sufficiently tested to allow of fair judgment.

Mr. Hutton's statements were of great interest to me. I have relied on unquestionable and well-known publications which he will have to correct. I may mention Trautwine, page 681. I am sorry that he has not mentioned his sources. The other errors mentioned are of less importance.

The bridge of 150-ft. span, near Ulm, is a foot bridge without hinges.

The remainder of the discussion I will divide into two heads, bridges and floors, and I intend, otherwise, to follow the statements of the authors as closely as possible.

Bridges.—Beginning with Mr. Gosling, I think he is correct, and my judgment of Coignet Béton should not have been given so abruptly.

The use of lime with Portland cement is, sometimes, so far as exterior finish is concerned, advisable with some kind of aggregates, but on account of a very regular abuse, I do not like it, and as there is no necessity for it, I prefer to be a little more particular about aggregates. For bridge work it cannot be recommended, and the composition called "Coignet Béton" does not keep volumes as well as Portland cement concrete.

Replying to Mr. O'Rourke, it seems well to state that a common concrete is not more able to stand the climate than any soft stone. If we remember how Nature has built sandstone and other aggregate stones, which is practically the same way as we build concrete, we know how to judge it.

The greater the pressure under which sandstone has sedimented, the less water remained, the less voids were left, the solider the stone, and the more fit is it to be used for exterior finish. Concrete should be judged in a similar manner. If we use the same concrete for exterior finish as we are accustomed to use for foundations or filling where much water and little ramming is necessary, we must fail.

Mr. O'Rourke gives an explanation of the cause of the shelling of the concrete, but I have strong doubts as to its correctness. If it really is possible, through change of temperature, to shell a concrete block, it would not be fireproof material at all, which, I think, nobody will doubt. The way this destruction of the surface occurs is by water which enters through the pores and freezes, as in any other stone. The shell will be just as thick as the frost can enter. This question is of even greater importance in harbor walls and with artificial stone of Portland cement, and I can say that it has been solved satisfactorily. It is only necessary to give the surface enough density not to allow the access of water, a similar process to that used in the construction of water-proof reservoirs and sewers of concrete. Grease, oil and some acids, which will be found not only in sewers, but also on the surface of the sea in harbors, are very detrimental to soft concrete. A dense surface will protect sufficiently, as the following test shows: Two cubes were prepared, the one of mortar, 1 to 3, and the other 1 to 1, and both were submerged in greasy oil. The first was destroyed, while the second hardened. I may refer to the perfect condition of a number of 20 year old oil tanks and sewers made of concrete.

There are many examples of the great damage which has been done by neglecting proper care of the exterior finish, especially in harbor work, where the worst weather and climate is joined with a great pressure of water, and, according to the tides, a continuous change of wet, dry and frost.

As an example of a failure could be mentioned the harbor of Aberdeen. Bamber & Carey on "Portland Cement," and Smith on "Portland Cement Concrete" (Minutes of Proceedings of the Institute of Civil Engineers, Session 1891–92), state that a lack of density of the surface is the only cause of the great damage which occurred there. All concrete work can be trusted to any reliable contractor, but to get out a good finish it needs something more than the experience of hearsay.

Much depends upon the choice of the aggregates as well as upon the amount of cement. Never use neat cement or a less mixture than

1 to 2½. The greatest density lies between 1 to 1 and 1 to 2½, and depends upon the quality of aggregates, which should be of all sizes, not dust-like, and the largest size in proportion to the thickness of the coating.

Regarding the inquiry of Mr. O'Rourke as to the amount of water used, I may say that, as a rule, concrete is strongest when made with the least amount of water. The amount can be proportioned in such a way that in ramming the concrete it shall show only a wet surface as a proof that enough water is in it.

A few words as to how to make an exterior finish.

There are two methods: First, to make common concrete and give it afterwards one or two coatings of dense mortar; and, second, to apply this coating inside while ramming the layers, and finish it up with a floating afterward. The method to be chosen depends upon location and other circumstances. It may be remarked, that, instead of a trowel, the use of a large wooden board is necessary, because it squeezes the water out equally and gives the coating an equal pressure. The coating must be well troweled in any case, so that the water rises to the surface, and floating and dusting continued until no more water appears. The second method, together with rubbing with a piece of felt, is generally used for artificial stone, and I may refer to the statue of Columbus at the World's Fair, and to the fountain, 20 years old, at the entrance of Prospect Park, Brooklyn, N. Y., as examples of the perfect finish which can be obtained.

The choice of material should not only be made by the eye, but always with sample tests.

An abstract of the report of Lieut.-Col. George L. Gillespie, Corps of Engineers, dated New York, N. Y., July 8th, 1893, is inserted with the permission of the author. He says:

"As a matter of professional interest, I desire to invite attention to the manner of constructing the counterscarp wall and galleries adopted by Lieut. Warren, to obtain a smooth surface, and one which is apparently weatherproof.

"The plan originally followed was to build all the concrete masonry in forms of undressed spruce lumber, and then, on removal of the forms, to plaster the exposed face with cement. This coat of plaster could not be made to adhere to the older masonry, which rapidly absorbed its moisture, weakening the bond between the two, which was destroyed by the first cold weather, when the coating cracked and dropped off, much to the injury of the appearance of the work.

"There was no economy in the use of spruce lumber, which warped out of shape when subjected to the moisture of the concrete, and could seldom be used more than once, and the plastering was very expensive.

"It was decided to substitute forms made of first quality dressed white pine, grooved on both edges, and united by loose tongues of yellow pine, and to construct the masonry as follows:

"That portion of the masonry next to the form, 4 ins. thick, was

put in without stone, the proportion of sand and cement remaining the same as in the concrete proper, securing a perfect bond between the face and the back of the wall.

"The forms being removed while the work was comparatively fresh, a little rubbing with a float gave a smooth surface to the face.

"The counterscarp wall and the galleries, constructed in this way, have withstood an unusually severe winter without scaling or cracking."

It is clear that this result was obtained by a little labor only, and that even better results are obtainable where the outside finish is more important.

But as in the Melan arch all iron is covered, I will not go deeper into this question. I have to state this definitely, because my designs do not show the coating which is supposed to be applied according to European practice, and are, therefore, somewhat misleading.

I may state further that English engineers in 1865, in building the West India Docks of London, thought that concrete could only be used as back filling, but they soon changed their minds, and in 1886 we read in the Minutes of Proceedings of the Institution of Civil Engineers, sessions 1886-87, a paper on "Concrete as Applied to the Construction of Harbors," that the use of every other material but concrete for harbor walls and dry docks belongs to the past.

I refer to the experience of the well-known firm of P. Ammann in Vienna, which has built, during 18 years' business experience, a very large number of water works, turbine dams and similar structures, and has always used only concrete walls with a concrete finish. If this was not satisfactory, they would certainly have changed their practice. Mr. P. Ammann states in a communication to me that he prefers a concrete finish on account of its low cost, as well as of its good quality,

My first impression was that Mr. O'Rourke's opinion was very common in this country, and consequently I intended to refer to much good concrete work which has stood for a long time in all weathers, but I soon desisted from so doing, especially as my attention has been called to the paper and discussion by Mr. Martin Murphy,* which covers the point.

Mr. Gayler asks whether an engineer is justified in reducing the dimensions to such dangerous limits. He will find in my paper that the safety maintained by those bridges is far above the safety usually given to iron bridges.

I do not feel inclined to discuss it further. I only desire to say that if an engineer would really use "his ingenuity to build the thinnest arch within the elastic limit," he would not deserve the name of engineer, but such an opinion should not be given without special proof. Whatever importance economical views may have, an engineer

* Trans. Am. Soc. C. E., Vol. xxix, p. 621, and Vol. xxx, p 567.

can only consider them after the engineering problem has been satisfactorily solved, as I have stated in my paper (page 454), giving also the economical question the last claim.

As this gentleman and Mr. Berger, as well as Mr. Hutton, express doubts, which on account of the space I have not answered fully, I submitted my paper to Prof. William H. Burr, of Columbia College, having been advised that he is one of the best authorities in this country regarding the theory of arch ribs.

The following is his statement:

"I have carefully examined the theory of the continuous arched rib with fixed ends, both under and independent of that given by Prof. Melan, on which the formulas of your paper on the Melan System of Arch Construction presented before the American Society of Civil Engineers, April 4th, 1894, are based.

"I find that those formulas are developed from correct theory, and, hence, that arches designed under them, with proper values of the empirical quantities which enter them, will be correctly proportioned and entitled to every confidence usually given to rationally designed engineering structures."

For further explanation it seems well to state that it is not an approved practice, as some of the gentlemen understood from my paper, to use concrete alone in tension. The calculations of concrete arches in Germany are generally based on the assumption that they are not subjected to tension by the safe live loads. The view taken in regard to the safety, and, consequently, to the breaking, load of such arches—loads which never occur, but are a scale on the reliability of the material—is quite another matter.

In a masonry arch the cohesion of mortar joints is taken as superfluous, and every part where tension occurs is practically supposed not to be a part of the arch. A stone arch must be a safe structure, with no mortar at all. Tension is taken into consideration for breaking loads, and the necessary safety can be easily attained.

I may refer to the tests made by Professors Engesser, Dyckerhoff, Pittel and Brautewetter on concrete arches alone.

It is good practice, and has been for a number of years, to allow tension in concrete when used in combination with iron.

The limit proposed by Prof. Melan, 60 lbs. per square inch, is the lowest to my knowledge, and according to the proportion of the elastic moduli the iron will have a tension of 2 400 lbs. per square inch. The concrete is not strained in two directions under maximum stresses, but for that case the stress resulting from both stresses is reduced to a lower limit of 42 lbs. per square inch. The spacing of beams is based on this supposition (see equation No. 3). While in the case of a concrete arch any improper connection in concrete will necessarily continue through the neighboring concrete, in this case it cannot show before the tensile limit of the iron rod keeping the concrete tight together is attained; that is, about 30 000 lbs. per square inch. We see

that the iron in this direction has a headroom of 13 in proportion to the stress which it is designed for. It is practically the same as if we had a stone arch connected at the joints by ties.

The omission of the action of the temperature was made in regard to the space which it would require and in regard to the fact that it does not need to be cared for in the spans of the length considered. For spans over 150 ft., hinges are to be recommended of the style Mr. Triest has described, and for the reasons given by Mr. Hutton.

The arch and the railing expand differently, and, if not specially designed and expansion joints provided, the latter will be cracked, as can be seen in every larger stone arch. I agree with the opinion of Mr. Hutton that this way of construction may have a great future and that it is superior to the usual arch with fixed ribs for larger spans.

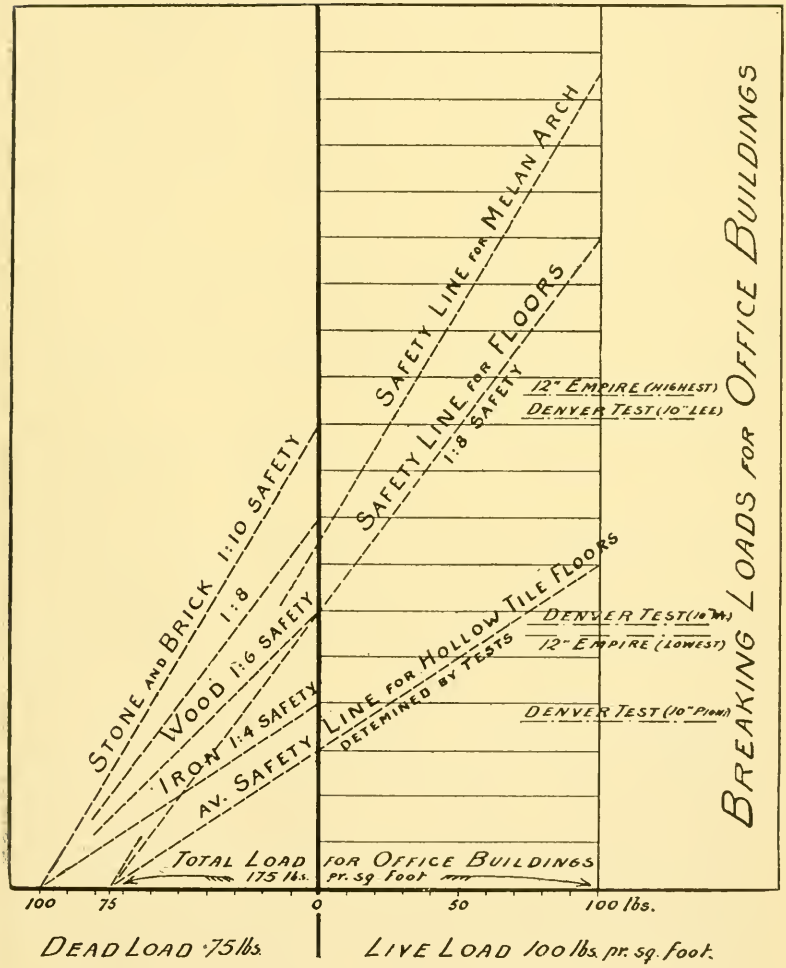
Mr. Gayler further says, regarding the Monier system, that "it will probably never be used for spans exceeding 50 ft., as it admits in no wise of a calculation of its strength." This is not well founded, as quite a number of Monier bridges have a larger span than 100 ft., and if he does not know how to calculate them, that is not a proof that we have not a quite reliable theory which is perfectly in accord with the tests. I have omitted the calculation regarding Monier and Wünsch, to reduce the size of my paper. I should think that tests of arches of 14-ft. and 77-ft. span, as made by the Austrian Society of Engineers and Architects, should cover the ground, especially in comparing this theory with those of iron bridges, the latter not always being backed by such full-sized tests. The specimens in all cases were made expressly for testing. Regarding the time, it is a fact that concrete improves, while this cannot be said of iron. A period of 10 years should be taken as a sufficient proof for an assertion of that kind.

The railroad crossing at Moedling has as heavy a traffic to carry as any other road. The dead load of concrete arches (as shown in Plate LXI) is 450 lbs. per square foot; the dead load of an iron bridge (Plate LX) 50 lbs. per square foot. Compared with the live load of 100 lbs., this gives a proportion of 9: 1: 2. It is consequently not bold, in my opinion, to claim that the former is free from vibration and tornado-proof, and the more so, as the connection with the supports is a far solider one than with iron bridges.

Regarding the tensile strength of concrete, I think it well to state that the Austrian Society of Engineers and Architects' standard does not call anything Portland cement, which gives in the proportion of 1: 3 after 28 days a tensile strength less than 200 lbs. per square inch, as an average of four samples. Quite a number of cement firms are willing to guarantee even a higher limit.

The strength of Portland cement concrete is equal to the tensile strength of the mortar, the cohesion between stone and cement being the same. But there is no reason to go to so high a guarantee if we want a concrete of 310 lbs. per square inch after three months. I think a

PLATE LXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 703.
VON EMPERGER ON CONCRETE-IRON BRIDGES.



guarantee of 150 lbs. sufficient, but would advise, in considering bids, not to consider the price only, but also the guarantee.

Wherever transverse and tensile tests of mortar are made together, their connection has been clearly proven.

Floors.—I will finally, in a few lines, touch upon iron concrete for floor construction.

A lively interest has been taken in its possible use for floors in fireproof buildings. This was unexpected, as my paper was intended, principally, to consider concrete iron as a bridge-building material. On this account, believing that the main features of the combination have been dealt with, I will confine myself only to an analysis of the comparison of a hollow tile floor and a Melan arch floor, as given by Mr. George Hill.

The diagram (Plate LXII) shows the present state of the floor question. I give only the figures of tests, which are, in my opinion, reliable as not being made by the manufacturer himself.

I. Tests with 12-in. block of Empire Fireproofing Company were made in 1891 by Mr. Cutshaw, City Engineer, Richmond, Va. They show a variation of the breaking load from 554 to 1 057 lbs. per square foot.

II. The well-known Denver tests, 1892, with 10-in. blocks of different make, showing a breaking load of 360 to 571 lbs. and 1 008 lbs., respectively.

From the diagram it will be seen that with my proposed reduction of safety to 1 : 8 an office building with 175 total load requires a breaking load of 1 400 lbs. per square foot, and as long as no tests with hollow tile can be produced showing these figures, Mr. Hill has no right to compare them with a Melan floor or any other floor (for instance, a wooden floor) which is constructed on correct engineering principles.

We meet people who are quite satisfied with a floor as long as it does not fall down, and I myself have seen hollow tile floors which could hardly have had a factor of safety higher than 2. It is certainly not sound engineering to advocate a method of construction on account of its cost only.

The following test was made last year at Brünn, Austria :

Three beams supported two 7-ft. spans, the two outside beams connected by heavy tie-rods and the middle beam left free to move. One span was then loaded with 1 400 lbs. per square foot for the whole surface. There were no signs of overloading, and the middle beam yielded only $\frac{1}{8}$ of an inch, and recovered afterwards when the load was removed, although free to move and without horizontal bracing. The distance between the rods in a common brick arch should not be more than 5 ft., and without tie-rods it would fall down at its earliest convenience. This is not the case with the Melan system. A proper factor of safety is necessary to the long life of a building. The Roman

concrete bridges could not have lasted 600 years if they had been so closely calculated to their load. Our iron bridges with a safety of 5 are not supposed to live more than two generations. What would happen if these figures are lowered further? Certainly our most splendid buildings could in that case not be called permanent.

Mr. Hill gives us a striking simplification of the method by which a Melan arch should be calculated, which is based upon his opinion that an arch can be calculated in the same way as a tie-rod. He considers the horizontal thrusts only. This leads him to say that the tension member (tie-rod) should have the same section as the arch itself. See his comparative table and his statement that "it will have the same amount of metal required as in a beam minus the web."

There is hardly any necessity to go further into Mr. Hill's theories. The same calculation of a hollow tile floor gives him a safe load of 770 to 1 027 lbs. per square foot. This means a breaking load of 7 700 to 10 270 lbs. per square foot, as he has used a factor of safety of 10. There are few tests which show a higher breaking load than his safe load, and this alone should warn us that there is something wrong in it.

I have given my opinion on this subject in a series of articles in the *Engineering Record*, under the heading "Testing Hollow Tile Floors," March 3d–31st, which also contains comparative designs for the table below.

The prices as given in Mr. Hill's table for the hollow tile floor I believe to be fair for New York prices, except that 5 cents per ½-in. leveling seems disproportionately large, but $18 90 per cubic yard for concrete, and $12 15 for leveling (ashes), in the Melan arch, must seem to anyone exorbitant prices, even for New York City.

Material.	Hollow Tile Floor.		Melan Floor.			
			6-ft. 8-in. span.		20-ft. span.	
Beams and girders	10.1 lbs.		9.2 lbs.		4.0 lbs.	
Connections	1.04 "		0.92 "		.....	
Tie-rods	0.3 "		.....		0.95 "	
		Cents.		Cents.		Cents.
	11.44 lbs. at 3 cts	34.3	10.12 lbs. at 3 cts.	30.4	4.95 lbs. at 3 cts.	14.9
Bent shapes	.....		1.8 at 3½	6.3	2.25 at 3½ cts.	7.9
Arching	.....	26	.....	14.	.....	20
Leveling	.....	2	.....	4.	.....	7
Cost of Construction per square foot	.....	62.3	.....	54.7	.....	49.8
Depth	16 ins. at 2 cents.	32	16 ins. at 2 cents.	32.	13 at 2 cents.	26
		94.3		86.7		75.9
			Flat ceiling	6.0		
				92.7		

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704.
(Vol. XXXI.—May, 1894.)

DREDGING OPERATIONS.—DISCUSSION ON PAPER No. 689.*

By ROBERT A. CUMMINGS, FOSTER CROWELL, J. D. VAN BUREN, E. B. GOSLING, F. A. WASHBURN, R. S. BUCK, R. L. HARRIS, CHARLES B. BRUSH and PETER C. HAINS.

ROBERT A. CUMMINGS, Assoc. M. Am. Soc. C. E. (by letter).—In all river and harbor improvements the dredging operations are perhaps the most important, and the appliances best adapted to the varying conditions should be very carefully considered. The work greatly depends upon the dredging operations; and, in this instance, the appliances used seem to indicate a very elaborate and interesting plant. It is, however, to be regretted that the author has not conveyed to us more information as to the dredgers and their mechanical efficiency, together with other details of operation. The length of time this work was in hand would make a detailed statement of the cost of the operations very valuable. Assuming the material was fine sand and mud, the location was very favorable for hydraulic dredgers.

In hydraulic dredgers, the height of the lift of the material is reduced to a minimum, and by using telescopic suction pipes the

* "Reclamation of the Potomac Flats at Washington, D. C.," by Peter C. Hains, M. Am. Soc. C. E.

dredgers would be independent of the variable height of the tide. The disadvantage of hydraulic dredgers, however, is that, in addition to the material lifted, they have to pump a greater percentage of water, which simply means a waste of power.

The author states that where "30 to 40% of the volume pumped is solid, such large quantities do not give good results"; this is undoubtedly the case where centrifugal pumps are used in combination with "agitators" or "mixers"; but does it not depend upon the class of material? It would not be hazardous to venture the opinion that vacuum pumps will handle the material in greater percentages and without being mixed by "agitators" or "mixers." The writer has designed such a hydraulic dredging appliance, and hopes to present, at no distant date, the result of experiments with the same. With centrifugal pumps, it has been found for silt and alluvial deposits 30 to 40% is the best proportion, and for fine gravel and coarse sand 10% of the volume pumped. The settling qualities of the material determine the best percentage to the volume pumped, and the pumping power should be proportioned accordingly. The percentage of solids pumped is obtained by taking samples from the discharge pipes and measuring the water and solids deposited in a graduated measure.

Another inherent defect of hydraulic dredgers is the impossibility of the captain knowing what the pump is doing; he has no positive knowledge whether he is pumping only water or a large percentage of solids; this feature is very important. The writer's improvements, referred to above, excavate a graded bottom by the self-feeding suction pipe. This is an advantage over ordinary hydraulic dredgers, which always leave the bottom full of very large pitholes and uneven places.

The author says: "The sand settled and packed in the discharge pipe, reducing its capacity." It would seem from this that the necessity of determining the settling qualities of the material in flowing water is necessary. The exceptional length of 3 000 ft. of the discharge pipe is, no doubt, accountable for a great part of the trouble. If the pump had been increased in speed or a relay pump placed about midway in the discharge pipe, the velocity of the water would have been greater, and the opportunity for settlement reduced to a minimum. It is not ordinarily economical to use the generally rough discharge pipes more than 1 500 ft. long.

Discharge pipes on pontoons are allowed to float about uncontrolled, having a number of slack lengths of pipe, to permit the movement of the dredger. These slack lengths of pipe often form sharp right angles to one another, owing to the unnecessary flexibility of the rubber joints. The waves from passing vessels or from wind raise one pontoon and its length of pipe above the next pontoon and its length of pipe. This strains the joint and often wrenches the fastenings so as to produce a bad leak, which causes expensive repairs and delays. The loss of head in discharge pipes due to eddies from swollen and uneven joints, joint-knees and excessive friction, are causes of much loss of power. An iron or wood discharge pipe floating on the surface of the water is a better arrangement. This can be accomplished by means of pairs of buoys connected by sling chains under the pipe and placed where necessary.

The author has referred to the velocity of the water flowing through the discharge pipe; it is hoped the method of determining the velocity of the water under pressure will be explained.

A statement of cost, with other details about the plant, interest, depreciation, repairs, coal, stores, etc., would be of much value.

In order to determine the mechanical work done, it is necessary to know the weight of the material and other conditions which may be given with advantage.

The author is to be congratulated on his comprehensive and instructive paper.

Foster Crowell, M. Am. Soc. C. E.—I should like to inquire more particularly about the prices. As stated in the abstract, the price for material handled twice by dredges and once in cars, and leveled by water-jet, was 21.2 cents, scow measurement, plus, I understand, 2 cents, the cost of trestle divided by the yardage; but there is a palpable error in the latter item. It should be 5.4 cents, if the figures of total cost of trestle and quantity of material moved over it are correctly given, making the total cost 26.6 cents; this, as I understand, is scow measurement.

Against it we have the results of the hydraulic dredges, given at about 14 cents, place measurement, with either pump used, which, on the basis of 20%, would correspond to about 11.5 cents, scow measurement, or considerably less than half, instead of a little more than half, as stated; that is, the difference appears to have been about 15 cents

a yard in favor of the hydraulic work, or, omitting the cost of trestle, about 9.75 cents per cubic yard saved by the hydraulic process. It is interesting to note that this saving would appear to be fairly represented by the second handling of the material by the plain dredge, but, as the combined lifts of the plain dredges were 60 ft., while the hydraulic dredge lifted only 27 ft., the economy of movement would be with the former; in other words, if the material had been deposited by the hydraulic dredges at a height within the reach of the plain dredges, the economy would have been on the side of the latter. This has a very important bearing on purely dredging operations where the material is to be moved afloat, and it would appear from these figures that, for scow work, with the character of material encountered on the Potomac Flats, the plain dredge would give the better results. Col. Hains' opinion on this point would be valuable.

John D. Van Buren, M. Am. Soc. C. E.—It may be of interest, in connection with the description of the dredging operations contained in the paper, to give the results of a successful operation recently completed in the city of Newburgh. Without attempting any formal description, I will describe as briefly as possible what we have done.

We have a lake, which constitutes our reservoir, of about 200 acres, and by the raising of the lake it was made to overflow extensive flats, aggregating about 50 acres.

The depth of the water before dredging varied from nothing to about 9 ft., and the depth of muck from 1 to 12 ft., with an average of about 6 ft. The result was that the water was very much injured, becoming almost unfit for use. It was decided to clean out these flats, and, in order to do so, without injuring or wasting the water during the operation, it was necessary to be very careful in choosing the method. Centrifugal pumps were excluded because they would require a stirrer to feed the suction, thus soiling the water, and wasting too much. The method finally approved by the engineers and adopted was proposed by Messrs. Babcock, Lary & Co., contractors of Newburgh, N. Y., who skillfully and successfully completed the work. The method adopted was this. A large scow was built, with a bin in the bow, about 10 by 20 ft., and about 4 ft. deep. A vacuum pump or chamber was placed upon the scow and fed by two boilers. The suction pipe led from the bin to the vacuum chamber with an intervening inlet gate. A discharge pipe led through the

scow, with a discharge gate at the chamber. A derrick, with a swinging boom, supporting a Heywood bucket, holding 2 yds. of material, was mounted on the bow with the necessary hoisting machinery. This discharge pipe was connected with a 15-in. pipe placed upon pontoons made of barrels and carried to the shore, where it was connected with the shore line leading to the dumps. We carried it to a distance of from 500 to 3 000 ft., with a maximum lift of about 44 ft.

The operation was this. The bucket lifted the material into the bin, the vacuum was formed in the usual manner, the inlet gate was lifted, and the charge filled the chamber. Then the inlet gate was shut, the outlet gate opened, steam was let in, and the muck was driven to the dumps. The material was almost wholly muck, varying from a consistency of peat to light, fibrous material.

In order to protect the water as much as possible during the operation, we put canvas curtains across the bays, and by that method we intercepted all circulation with the main body of water. By this method we removed about 400 000 yds.

Comparison has been made in the paper between the scow measurement and measurement in place. The result in this case was, the bucket measurement, which we tallied, was some 15 to 17% less than the measurement by cross-section; that is, the muck occupied more space in the bottom of the lake than it did in the bucket. We found this dredge was not so well calculated to remove stiff clay; probably it would remove sand and material of that kind. As to the additional quantity of water used, we found we could reduce it to about 15 per cent; that is, the quantity of solid material discharged was about 85% of the total discharge passing the pipe.

The operation was a complete success, and we carried it through without injuring the water in the slightest degree. The water has been good since the completion of the work of last year—which formed the greater part of the excavation.

Mr. Crowell.—What was the cost?

Mr. Van Buren.—It was done by contract, and the bid was 28 cents; we had bids by prominent firms ranging as high as 50 cents; the lowest was 28.

H. W. Brinckerhoff, M. Am. Soc. C. E.—What was the pressure?

Mr. Van Buren.—The pressure was about 50 lbs. We had some trouble from the stones occasionally lifted, or old trunks of trees, or

something of that kind, but not much. There was considerable trouble at first with the flexible india-rubber joints of the pipe line, but by having them made with spiral-wire wrapping, the trouble ceased.

Edgar B. Gosling, Jun. Am. Soc. C. E.—I have seen dredges discharging through pipes on pontoons in which the difficulty of the breaking of the joints, just mentioned, was overcome, to a great extent, by having the pontoons connected by chains, which did not allow the angle made by the pipes to change but a small number of degrees and thereby not strain the joints as much as where such chains were not used. That kind of discharge was used where the material had to be spread over large surfaces. At the same time there was dredging going on close by (this was, I may say, at the Suez Canal) where the endless-chain dredge, with a floating structure with a large discharge pipe or chute carried by means of a cantilever, was used. I have seen these cantilevers, from 200 to 300 ft. long, discharging the dredged material on the sides of the canal quite a distance in shore, so as not to bring undue pressure on the banks. It was considered economical to use the hydraulic dredge and floating pipe discharge for distances from 1 000 to 2 000 ft. I think the pipes used were larger than ordinarily used here, say, about 18 ins. in diameter.

F. S. Washburn, M. Am. Soc. C. E.—The author speaks of 40 to 60 ft. as about the average to which the material was lifted in using ordinary dredges and cars. Of course that is an objectionable feature, but the cost of an extra foot in height in excavating material after it is once gotten under way, is in many instances inappreciable. I have endeavored to estimate, as I have been sitting here, listening to the paper, about what it would cost, under ordinary conditions of dredging, to raise 1 cu. yd. of material that extra 40 to 60 ft., and it is somewhere between $\frac{1}{3}$ and $\frac{2}{3}$ cent a cubic yard. There may be advantages in the ordinary type of dredge, compared with which this extra cost of handling material may be relatively unimportant. In a number of instances I have known devices argued against and thrown aside because the amount of actual mechanical work necessary was greater than in some other method with which it was compared, but which possessed practical advantages entirely outweighing its low efficiency. The point I make is that, while the mechanical work may be much greater, the actual extra cost due to the extra work may be of no consequence, or very small.

R. S. Buck, Assoc. M. Am. Soc. C. E.—Since this discussion has developed the question of the relative efficiency of the different kinds of dredges, I would like some gentleman to give us some information as to the relative efficiency of the dredges as actually lifting the material from place. When I was on work in Vicksburg Harbor under the Mississippi River Commission some time ago that question arose, particularly as to the comparative efficiency of the endless-chain and the clam-shell dredges, and while I had at the time no fair means of getting at the efficiency of the endless-chain dredges, I got pretty well what was the actual cost of raising material from place, and depositing in the river again by means of dumping scows, at an average distance of about a mile and a half. I found the actual cost was about 5 cents per cubic yard. The average depth of water, about 35 ft., I think; the cost was about 5 cents per cubic yard, including towing.

I would like to know, as a matter of curiosity, what the cost of dredging under known conditions with the endless-chain dredger is.

Mr. Crowell.—I happen to know of a case where 10 cents per cubic yard was said to be the price paid to a contractor using the endless-chain dredge. The endless-chain, I think, is more economical than the dipper dredge in working large prisms. There is a point where the dipper dredge or clam-shell dredge begins to lose its economy very rapidly as the depth increases; but in moderate depths, and where the operation is simply to lift the material upon scows, I believe the large-sized dipper dredge will do the work for the least money.

Mr. Gosling.—Is it not more difficult to get an even bottom with the dipper dredge than with the endless-chain dredge?

Mr. Crowell.—Yes, sir.

R. L. Harris, M. Am. Soc. C. E.—In counting that cost of dredging at 5 cents, did you take into account the deterioration of plant, the interest on the cost of outfit, and the item of lost time?

Mr. Buck.—No, sir; I did not count the deterioration; merely the cost of operation.

Mr. Harris.—Neither the deterioration, nor interest, nor loss of time?

Mr. Buck.—That included repairs; it was simply the cost of operation.

Mr. Harris.—I may say that the author speaks of 863 000 cu. yds. and 557 000 cu. yds. having been deposited from the trestles, and then the pumping business was begun. Now, the original idea of one of the

figurers on that work was the pump and the figures he made—14 cents—were almost identically those of the actual cost by pump.

Charles B. Brush, M. Am. Soc. C. E.—That was before any work was done at all ?

Mr. Harris.—Yes, sir; at the time of the original bidding for letting.

Mr. Brush.—After that, the plan was changed and the trestle was built.

Mr. Harris.—There was no limit, as I remember, to what plan could be used. One of the figurers figured entirely on the use of pump.

Mr. Brush.—Was the work let out by contract originally ?

Mr. Harris.—That is my recollection of it.

Mr. Brush.—Then the lower bid was thrown out ?

Mr. Harris.—I do not say anything about bids; I say this was the estimated cost by one of the figurers.

Mr. R. S. Buck (by letter).—Since the date of the discussion of Col. Hains' paper, I have received the following letter concerning cost of dredging in Vicksburg Harbor.

"United States Engineer's Office,
Vicksburg, Miss.,
January 2d, 1894.

Mr. R. S. Buck.

Dear Sir,—In answer to your favor of the 27th of December, requesting information in regard to cost of dredging in Vicksburg Harbor, I have the following to state: I find in my report to Captain Rossell of May 2d, 1888, the cost of operating the contractor's dredge, *Herndon* (clam-shell bucket), to be for 24 hours:

16 men (laborers), each, $30 per month	$16 00	
2 engineers, each, $60 per month	4 00	
2 captains (diggers), each, $125 per month,	8 25	
2 cooks, each $45 per month	3 00	
Total—dredge		$31 25
Tug, 6 men, each $30 per month	$6 00	
" 2 captains, each, $125 per month	8 25	
" 2 cooks, each, $45 per month	3 00	
" 2 engineers, each, $60 per month	4 00	
Total—tug		21 25
Subsisting 36 men (including two U. S. Inspectors), at 50 cents		18 00
Coal for dredge and tug, 6 tons per 24 hours at 40 cents per box		25 00
Incidentals, repairs, etc.		10 00
Total cost		$105 50

An average month's excavation (double turns) was 55 529 cu. yds., (scow measurement), giving a little more than 5½ cents per cubic yard as actual cost of dredging. This does not include interest chargeable to money invested on plant or cost of getting to working locality. An estimate made in 1883, in Captain Marshall's time, gives the same results.

Yours truly,

H. St. L. Coppée."

I would like to add further, that I furnished the data for this estimate, which, on account of the exact rate of wages paid all the men and the exact cost of subsistence not being known, is only an approximation ; but the tendency of the figures given is undoubtedly to make the cost of operation appear somewhat larger than was actually the case.

Now, valuing the entire plant at $50 000, which is high, interest at 6% would be $8 22 per day. A considerable portion of depreciation is included in the item in Mr. Coppée's letter, "Incidentals, repairs, etc."; but taking this at 10%, it gives $13 70 per day. These items with the $105 50 accounted for in Mr. Coppée's letter will give a total daily cost of operation of $127 42, or a cost per yard of something less than 7 cents. This does not account for towing to and from place of operation, which item cannot well be estimated in a way to aid comparison.

The digging in Vicksburg Harbor might be called "fairly good." The bucket was rarely heaped, and some very tough material and very large logs were encountered and removed. The *Herndon* was ably handled and worked with great energy.

I think the above will furnish fair means of comparison as far as the clam-shell dredge is concerned, and I believe it would be of general interest if similar data could be had as to the performance of dredges of other types.

P. C. Hains, M. Am. Soc. C. E.*—It is proper to say that the hydraulic dredges used on the reclamation of the Potomac Flats were not designed by me, and I claim no credit for their efficient working and am not responsible for defects. That there were many defects in the machinery is unquestionable. But the particular interest that attaches to this work lies in the fact that it was the first application

* Lt.-Col. Corps of Engrs., U. S. A.

known to me of the hydraulic dredge to a work of magnitude. It was not the first application of such a dredging machine, for Colonel G. H. Mendell, of the Corps of Engineers, had one in operation at or near Oakland, Cal., before the work on the Flats was begun.

The question of reducing the number of angles or bends in the pipes, as well as the necessity of devising some better means of connecting them, was a frequent topic of discussion between me and the contractor. I do not regard the hydraulic machine used by us on the Potomac as the perfection of hydraulic dredge. I think there are many things in connection with it that are susceptible of improvement, and as the machine comes more generally into use, improvement will follow.

The velocity of the water flowing through the pipes was never determined with any degree of accuracy. It was rather roughly estimated from observation and the known capacity of the pump working under certain known conditions.

In regard to the prices of the material handled twice by the dredges and once by the cars, and leveled by the water-jet, it should have been stated that 21.2 cents per cubic yard was the price paid by the Government for the dredging and depositing of the material in place at the required grade. It included the cost of the trestle, cars, locomotives and everything, including the profits to the contractor, if there were any. What I meant to call attention to was, that the trestle alone cost the contractor, not the Government, about two cents for every cubic yard of material he handled, the amount of material handled being nearly 1 500 000 cu. yds., and the cost of the trestle about 3 000 000 cents ($30 000). With this explanation the apparent error referred to by Mr. Crowell disappears.

As to the comparative cost of dredging by the hydraulic machine or by the ordinary dredging machine in common use, such as the clam-shell and dipper dredge, it is impossible to say what that will be until a full understanding of the conditions is assured. The ordinary clam-shell or dipper dredge is adapted to many varying conditions; the hydraulic machine is not. On the Potomac Flats the material had to be taken out of the bed of the river, transported from 1 000 to 5 000 ft., and then deposited in places where it would be useful as earth filling. It could not be carried in scows to the places of deposit, for there was not water enough to float them, and most of the material had to be de-

posited above the level of the water in the river. Some means of transporting the material had to be improvised, and the use of water flowing through pipes at a high velocity was the most economical.

To make a fair comparison between the cost of the two kinds of dredging one must know exactly the character of the material, the depth of cutting, the exposure of the locality to storms and waves, the distance to which the material, after being dredged, has to be taken, and the relative height of the place of deposit. If it be a simple case of dredging and depositing the spoils in a scow, the ordinary clam-shell or dipper dredge would give the most economical results as a rule, but not always. Much will depend on circumstances, such as the exposure of the locality and the character of the material. The sandy bars at the mouths of rivers exposed to the open sea are generally so rough that a hydraulic machine provided with a flexible pipe reaching to the bottom gives far the best results. At the entrance to New York Harbor, the hydraulic machine was eminently successful, and the ordinary clam-shell or dipper dredge a failure; so, also, at the mouth of the St. Johns and Cape Fear Rivers, and other places. To institute a fair comparison between the two methods, the conditions must be taken into consideration. In the case of the Potomac Flats the conditions were peculiarly favorable to the hydraulic machine.

The point made by Mr. Washburn is well taken, that, while the mechanical work may be much greater, the actual cost due to the extra work may be of no consequence, or very small. That is just what operates in favor of the hydraulic dredge. The extra mechanical work of lifting and forcing an immense quantity of water through pipes is great, but the extra cost is comparatively small.

I may also add, that in offering the work to contractors, the method of doing it was always left to be stated by the contractor himself, it merely being required that the method should be practicable and adapted to its accomplishment. The first contract was awarded at 21.2 cents per cubic yard, scow measurement, that being the price given by the lowest responsible bidder. There was one other, and only one other, bid made at the time that gave a lower price, and that was 19½ cents per cubic yard. The bid, however, was informal and defective, the bond worthless, the bidder without the means of doing the work in any manner whatever, and not responsible. That bid was accordingly rejected.

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705.

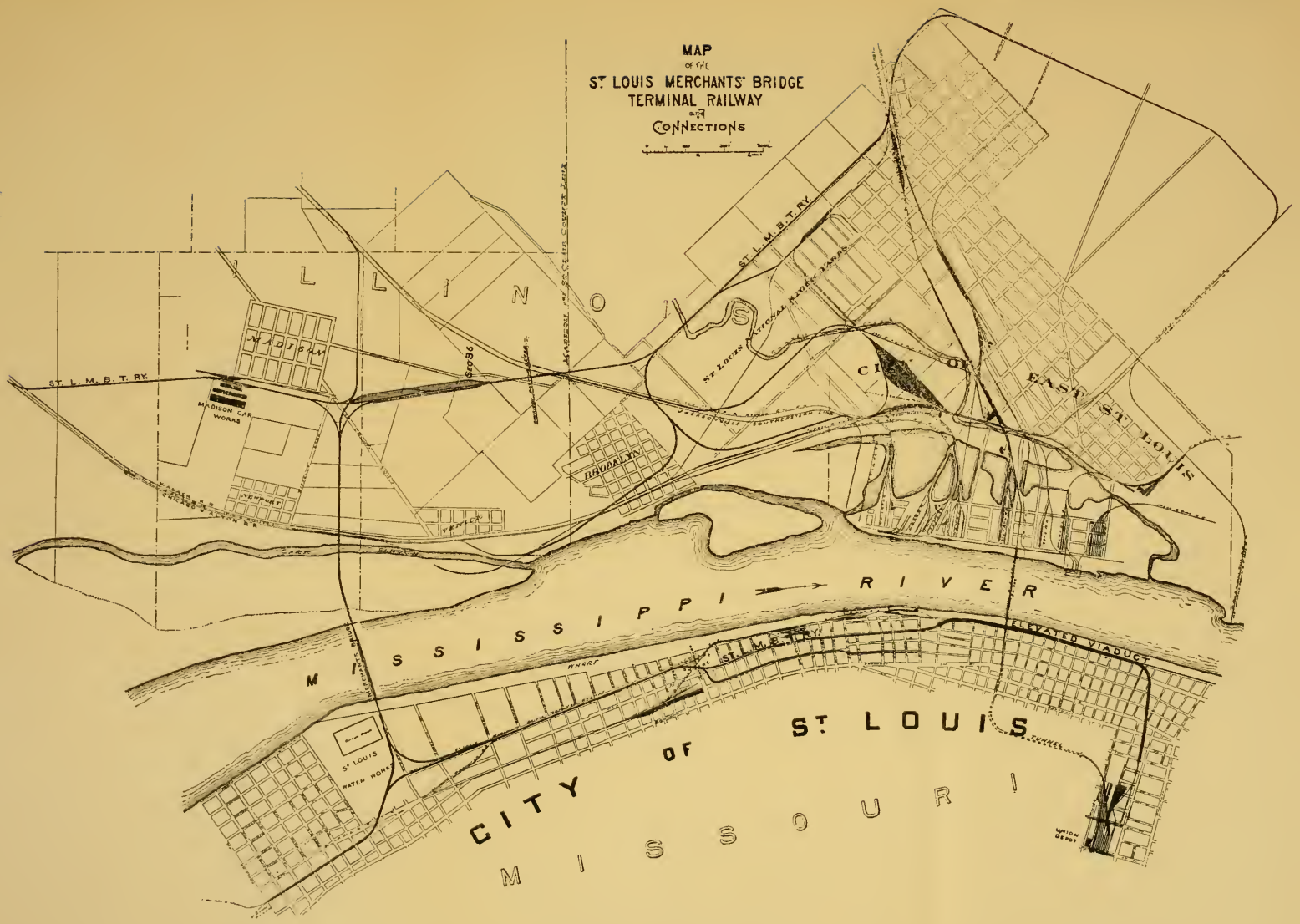
(Vol. XXXI.—May, 1894.)

THE MERCHANTS' BRIDGE TERMINAL RAILWAY VIADUCT AT ST. LOUIS, MO.

By ROBERT MOORE, M. Am. Soc. C. E.

In developing a new system of terminal tracks designed to unite the various railway systems of St. Louis with the Merchants' Bridge over the Mississippi River it became the duty of the writer as Chief Engineer to build a section of elevated railway which it is the purpose of this paper to describe.

This structure was authorized by Ordinance 14 078, approved July 9th, 1887, and had for its special object to reach the south end of the city and the railways in the Mill Creek Valley. As will be seen by the accompanying map (Plate LXIII), it extends from the north side of Carr Street, over which it passes, through city block 18 to the river front, along which it runs southwardly for more than a mile to Poplar Street. Here it curves sharply to the west and passes through the blocks to the west side of Seventh Street about 100 ft. north of Gratiot Street, where the permanent iron structure ends. Beyond this, 150 ft. of temporary wooden trestle brings the track to the surface of the ground, and in a few hundred feet more it is connected with the tracks of the St. Louis and San Francisco and Missouri Pacific railways. The total length of iron structure is 8 $176\frac{7}{10}$ ft., or about $1\frac{55}{100}$ miles.


MAP
OF THE
ST. LOUIS MERCHANTS' BRIDGE
TERMINAL RAILWAY
AND
CONNECTIONS
ILLINOIS
ST. L. M. B. T. RY.
MADISON CAR WORKS
ST. LOUIS NATIONAL STOCK YARDS
EAST ST. LOUIS
MISSISSIPPI RIVER
ELEVATED VIADUCT
ST. LOUIS WATER WORKS
CITY OF ST. LOUIS
MISSOURI
UNION DEPOT

The preliminary plans were made during the summer of 1889, and in November of the same year a contract for the construction of both foundations and superstructure was made with the Phœnix Bridge Company. Work was begun in December, 1889, and the whole structure completed, excepting the painting, in March, 1891, about fifteen months from the time of beginning. This was longer than would have been necessary but for delays in securing the right of way, such as appear to be almost unavoidable in works of this kind—delays which were a source of embarrassment to the engineer as well as to the contractor. To avoid enhancing the price of real estate needed by the railway company, which would have resulted from the appearance on the ground of an engineering party, no such party was put into the field until all property that could be bought at private sale had been secured. The preliminary plans had, therefore, to be made from such imperfect data as could be obtained from city maps without a survey. In some cases exact lengths of girders could not be safely given until buildings which stood in the way had been torn down so as to allow of actual measurements on the ground. Even where there was no lack of exact knowledge, the fact that the contracts for right of way nearly always contained some stipulation as to the location of the columns rendered it impossible to make a final plan until all such negotiations were definitely closed.

The details of grades and curvature are shown on the accompanying profile (Plate LXIV). The sharpest curve is one of 14° (410 ft. radius) for about 260 ft. There is also one curve of 12° (478 ft. radius) for 160 ft., and one of 10° (574 ft. radius) for 60 ft. The remainder range from 2° 45′ down to 1°. All the longer curves are connected to the tangents at each end by spiral transition curves. The maximum rate of grade is $\frac{8}{10}$ of 1%, or $42\frac{24}{100}$ ft. per mile. On curves the grade is reduced half a tenth per 100 ft. per degree of curvature, and all changes of grade are made by vertical curves. Going southwardly the length of descending grades amounts to $28\frac{4}{10}$%, the level grades to $28\frac{6}{10}$%, and the ascending grades to 43%, of the whole line.

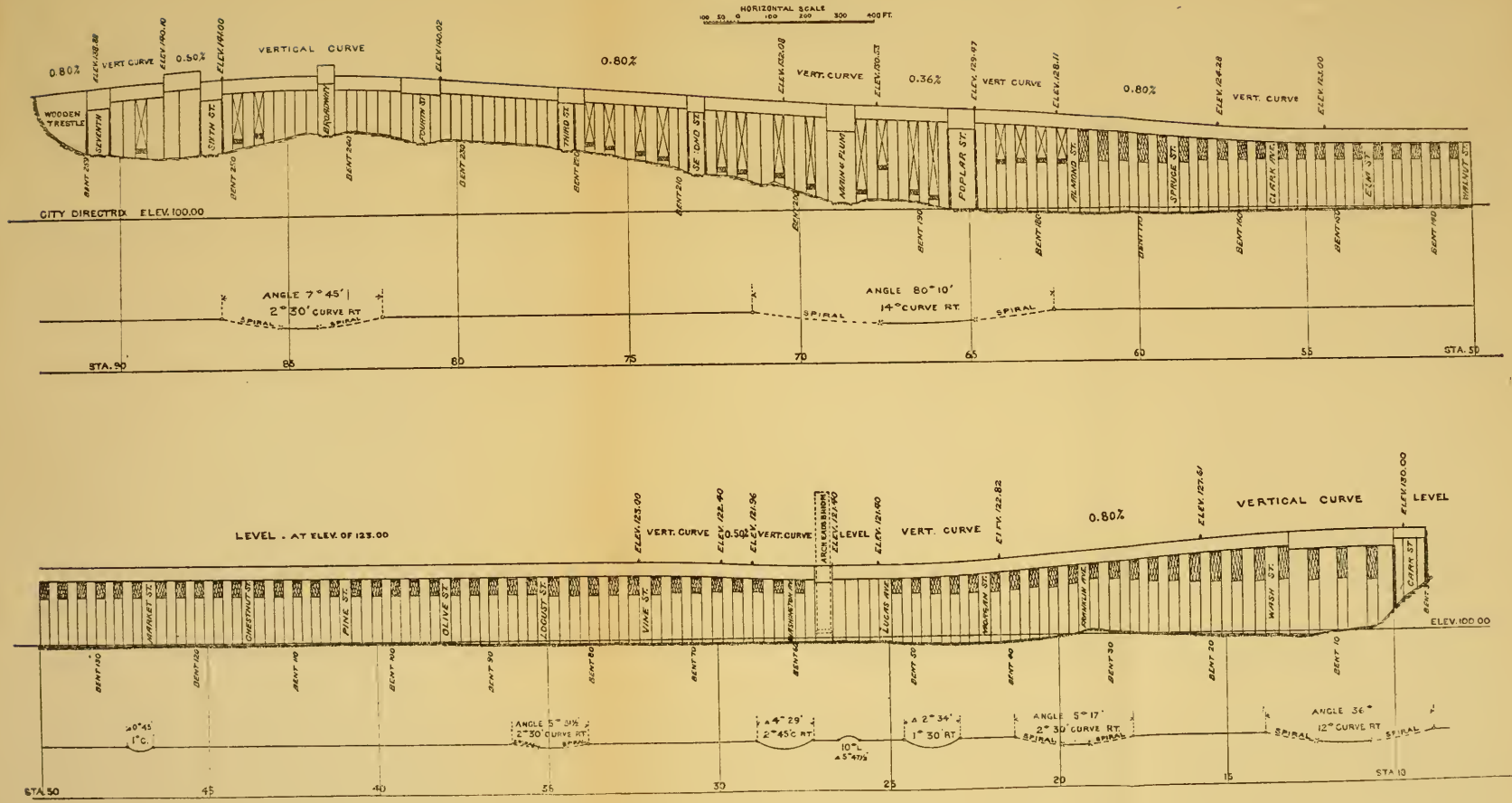
The structure itself consists of four lines of plate girders carrying two lines of railway track. With the exception of twelve masonry piers all the supports are steel columns built of Z bars or of plates and angles. The two rows of columns are directly in line with the two outer girders. This brings them $18\frac{1}{2}$ ft. apart between centers and allows just space

enough for two lines of teams, thus causing the least possible interference with the traffic on the street. The height and section of the column is made such as to allow both the longitudinal and the cross girders to be riveted to them directly, which is done everywhere except at expansion joints and in a few cases where special conditions made it possible. To further stiffen the structure alternate bents are braced, where it is practicable, as in passing through the blocks, by struts near the ground and diagonal rods. When this method would interfere with the street traffic a partial bracing is substituted, consisting of latticed struts brought down to a line 14 ft. above the pavement.

The column foundations are all formed of Portland cement concrete surmounted with granite cap stones. Imbedded in the concrete and extending through the cap stones are anchor rods to secure the columns to the foundations. In those parts of the structure where the bending stresses are taken up by bracing brought down to the ground, the only duty of the anchor rods is to prevent displacement at the foot of the column, for which light rods only are needed. Those used are $1\frac{1}{4}$ ins. in diameter, 3 ft. long, two to each column. When the bracing is not brought down, the bending stresses are transmitted to the anchor rods, and they were therefore proportioned with a view to holding the column at the bottom rigidly "fixed ended." For this purpose four rods $1\frac{5}{8}$ ins. in diameter, with upset ends, were used for each column. Their lengths was usually $7\frac{1}{2}$ ft., though in special cases they were made longer. In deference to the common fear of the injurious effect of cutting screw threads in steel, the anchor rods were made throughout of iron. The result was not satisfactory. Upon testing full-sized rods, all of them broke in the screw thread, showing, apparently, a weakening of the metal in the process of upsetting. After annealing, however, their strength was so far restored that, upon further tests, they broke in the body of the bar. They were, therefore, accepted and placed in the structure. Two of them, however, were broken while in position during construction, and were the only failures which at any time occurred. Had they been of steel, this fact would have been accepted as a sufficient explanation of the failure.

In placing the anchor rods use was at first made of wooden measuring rods furnished by the Phœnix Bridge Company, and certified by them to be in exact agreement with their shop standards. It was soon found, however, that the practical difficulty of making and main-

PROFILE
OF THE
MERCHANTS BRIDGE TERMINAL RAILWAY VIADUCT
AT
ST. LOUIS



taining perfect contact between the rods and of keeping them at all times level was so great as to make the results less trustworthy than those obtained with a steel tape, which was thereafter used to the exclusion of everything else. The results thus obtained checked so well with the shop measurement that in setting nearly 2 000 anchor bolts for more than 1 200 longitudinal and cross girders no misfits of any consequence occurred.

The material encountered in the foundations was mainly a firm clay upon which a maximum load of 3 000 lbs. per square foot including the live load, was placed. Ordinarily a much greater load no doubt could have been safely borne, but the fact that in times of freshet all the foundations on the river front are under water and liable to saturation made it desirable to keep the loading as low as reasonably practicable. In some cases where quicksand or soft mud was encountered piles were used, over which the concrete was laid without capping, as shown in the plans.

In eighty-three cases the foundations were placed directly upon the rock. In a few of these the rock was found so near the surface that holes had to be drilled to receive the anchor bolts and the question arose as to how to fasten them so as to develop the strength of the bolts and obtain an anchorage equivalent to that of the block of concrete used in other cases. To solve this question two sets of experiments were made. In the first, a number of holes were drilled in a solid ledge near the line of the work, in which holes were inserted rods fastened by means of wedges, iron cement or "rust joint," sulphur, lead and Portland cement used neat and also mixed with sand. By means of an I beam arranged as a lever and weights of known value, a pull was then applied to each bolt until it was either pulled out or broken off. As the result the bolts fastened with wedges and with iron cement were all easily pulled out. Of those fastened with lead and sulphur, each melted and poured in around the bolt, two out of three were broken and one pulled out. Of those fastened with neat cement five out of six were broken and but one pulled out. The two bolts fastened with a mortar of cement and sand in equal parts were both broken.

These results showed so well for the cement anchorage that another series of tests with neat cement alone was made at the Washington University testing laboratory. In this series holes were drilled in limestone blocks 10 x 10 ins. square and 12 ins. deep, this being as

large blocks as could be got into the testing machine, and in these holes, rods of 1 and 2 ins. diameter were fastened with a mortar of neat Portland cement. Half of the rods were plain, the other half had a screw thread running the full depth of the hole. Ten days were allowed for the cement to set, and the rods were then pulled until loosened in the holes or until the stones were broken. The plain rods developed before loosening a resistance of about 500 lbs. per square inch of imbedded surface. The threaded rods showed a still higher resistance, in fact the stones were not strong enough to develop the full strength of the anchorage. To bring out all the facts a further series of experiments with larger stones would be required, but these results were so satisfactory that wherever anchorage in rock was required, threaded rods fastened with neat cement were used.

The concrete used in all foundations was made of Portland cement, river sand and broken limestone in the proportions of 1 part of cement to 3 of sand and 6 of stone. With these proportions, it was found that one barrel of cement made 0.96 yds. of concrete. To ascertain the strength of this concrete, a number of beams 30 ins. long and 9 ins. square were made from batches mixed for actual use in the work. These were then buried in the ground for periods of 8, 9, 10 and 11 months. At the end of this time they were broken transversely, by means of a lever apparatus so arranged as to give a gradually increasing, but definitely known, load. The results are shown in the table on page 505.

The tensile strength computed from these experiments showed an unexpected uniformity, the average being 354 lbs. per square inch, with a minimum of 307 and a maximum of 412 lbs. The strength of the neat cement from which the concrete was made, as shown by a seven-day test, ranged from 307 to 567 lbs., the average being 432 lbs. per square inch. That is to say, that while the variation in the strength of the neat cement was 85%, the variation in the concrete was only 34 per cent.

With the exception of the anchor bolts and a few minor members, the material used is mild steel with an ultimate strength of not less than 60 000 nor more than 68 000 lbs. per square inch, an elastic limit of not less than 37 500 lbs., and an elongation in 8 ins. of not less than 22 per cent. All rivets were made of soft steel having an ultimate strength of not less than 54 000 nor more than 62 000 lbs. per square

inch, an elastic limit of 33 750 lbs., and an elongation in 8 ins. of not less than 25 per cent. For both grades of steel the usual quenching and bending tests were also specified, but no chemical tests of any kind were required. All sheared plates were required to be planed and all rivet holes reamed.

Cement Tests.				Concrete Tests. Proportions, 1, 3 and 6.							
Brand of Cement.	Time, in days.	Mixed Neat, pounds.	Mixed 1 to 1, pounds.	Number of beam.	Age, in months.	Dimensions in inches. Depth.	Width.	Length.	Breaking load.	Computed tensile strength.	
A	7	307	195	1	8	9¼	9¼	27	6 610	338	Pounds per square inch.
A	28	502	440	2	9	9	9¼	27	7 290	394	" " "
				3	9	9½	9	27	6 520	325	" " "
							Mean value.....			352	
B	7	503	299	1	9	9¾	9	27	6 490	307	Pounds per square inch.
B	28	591	355	2	8	10	9½	27	8 030	342	" " "
				3	9	9	9¼	27	7 160	387	" " "
							Mean value.....			345	
C	7	567	320	1	9	9	9½	27	6 500	342	Pounds per square inch.
C	28	703	482	2	9	9¼	9½	27	7 470	372	" " "
				3	9	10	9¼	27	7 980	349	" " "
							Mean value.....			354	
D	7	350	197	1	10½	9¾	9	27	7 610	360	Pounds per square inch.
D	28	545	338	2	10½	9	9	27	6 030	335	" " "
				3	11½	9	9½	27	6 780	357	" " "
				4	11½	9	9½	27	7 820	412	" " "
							Mean value.....			366	

The superstructure was proportioned for a dead load consisting of the weight of the iron and steel in the structure itself, plus the weight of the track, assumed as 450 lbs. per lineal foot, and a live load for each track, assumed to be moving in either direction and consisting of two engines coupled, each with its tender weighing 208 000 lbs., being Mr. Cooper's "Extra Heavy A." The longitudinal bracing was designed to resist the force developed by suddenly stopping a train of 3 000 lbs. per lineal foot, headed by two of the engines just mentioned, the coefficient of friction of wheels upon rails being assumed at 20% of the load on the wheel. The wind pressure for computing the lateral stresses was taken at 750 lbs. per lineal foot.

In designing girders, a shearing stress was allowed of 7 000 lbs. and a flange stress of 10 000 lbs. per square inch of net section. Wind bracing and anchor rods were proportioned at 15 000 lbs. per square inch, the stress on anchor rods being increased by 10 000 lbs. for initial adjustment.

In designing the posts of the partly braced structure, the unit stresses were varied for the three different cases as follows:

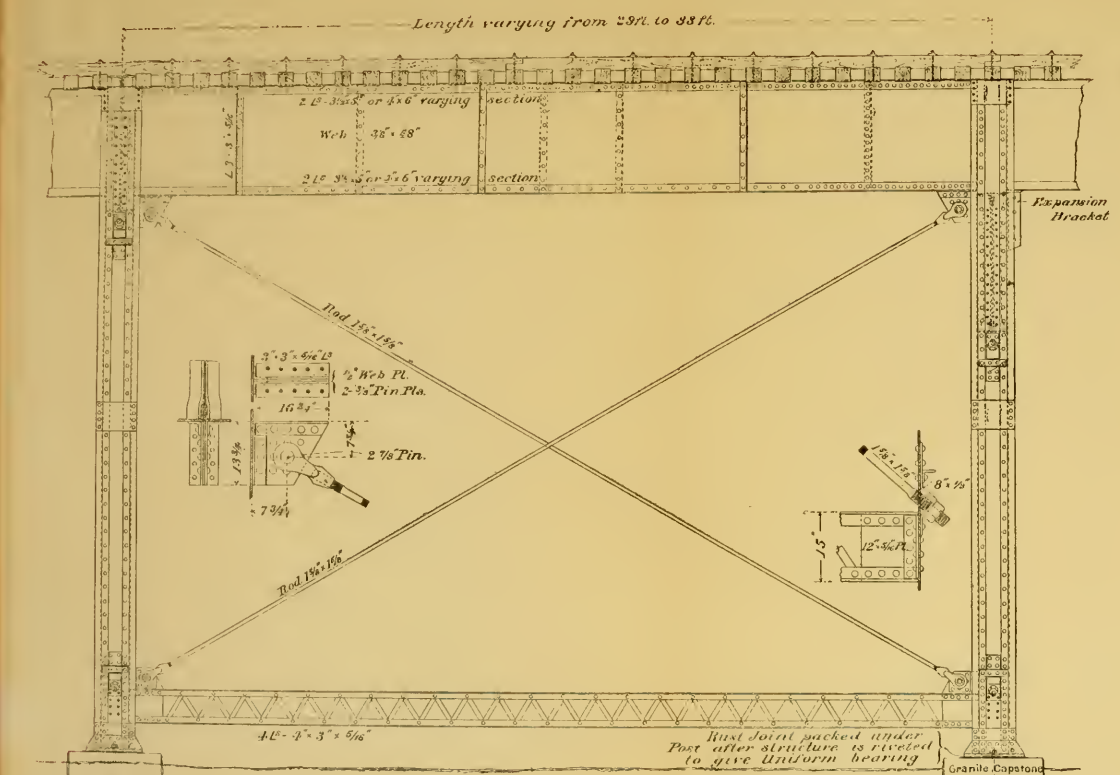
First.—For the direct load and bending due to the tractive force of the engine, this being a working condition, a unit stress of 10 000 lbs. per square inch was allowed.

Second.—For the direct load and bending due to wind pressure or a sliding train, an exceptional condition, a unit stress of 14 000 lbs.

Third.—For the direct load combined with bending due to wind and tractive force or a sliding train, also an exceptional condition, 20 000 lbs. In each of these three cases it was found that the amount of metal needed to resist bending was much greater than that required for the direct load. The total weight of iron and steel in the structure is 9 791 000 lbs., or 1 197 lbs. per lineal foot.

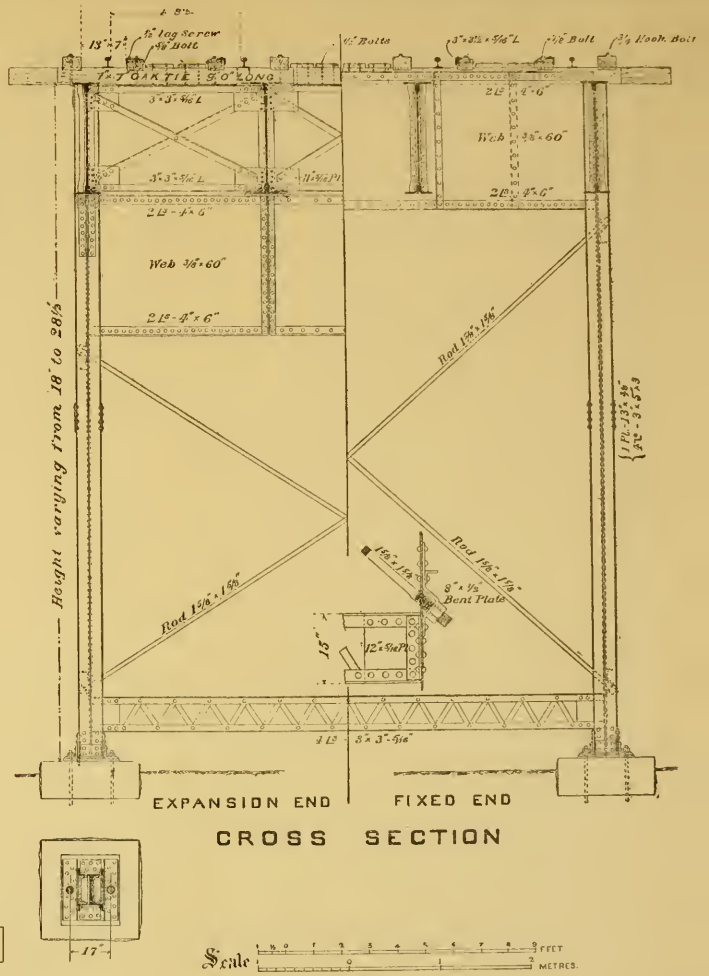
With the exception of five or six spans, the whole structure was erected by means of a traveler moving on a temporary track laid on the outer girders. The material was brought to the site of the work by teams and lifted to final position by the traveler. When not delayed for material, six spans were easily erected in one day of 10 hours. Before the work of erection had proceeded far it was found that the bearing of the columns on the granite cap stones of the foundation was not as true and uniform as was desired. This was due sometimes to the fact that the cap stones were not a perfectly level plane, and at other times to the fact that the base plate of the column was not a true plane, or not exactly perpendicular to the axis of the column. As this want of uniform bearing would develop unequal reactions at the base and eccentric stresses in the column itself, it was necessary to correct it. After unsuccessful experiments with Portland cement and with lead, it was found that by far the best method was to lift the column slightly by wedges, say $\frac{1}{4}$ in., and then fill in the space thus left between the cap stone and the base of the column with iron cement or "rust joint." By using long blades of thin steel and borings of cast iron, this was easily done, and when well done gave an entirely satisfactory result. Upon taking down a post, referred to

PLATE LXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 705.
MOORE ON MERCHANTS' BRIDGE TERMINAL RAILWAY.



SIDE ELEVATION

THE MERCHANTS BRIDGE TERMINAL RAILWAY VIADUCT
—AT ST. LOUIS—
TYPICAL PLAN OF STRUCTURE
WITH FULL BRACING



CROSS SECTION

Scale

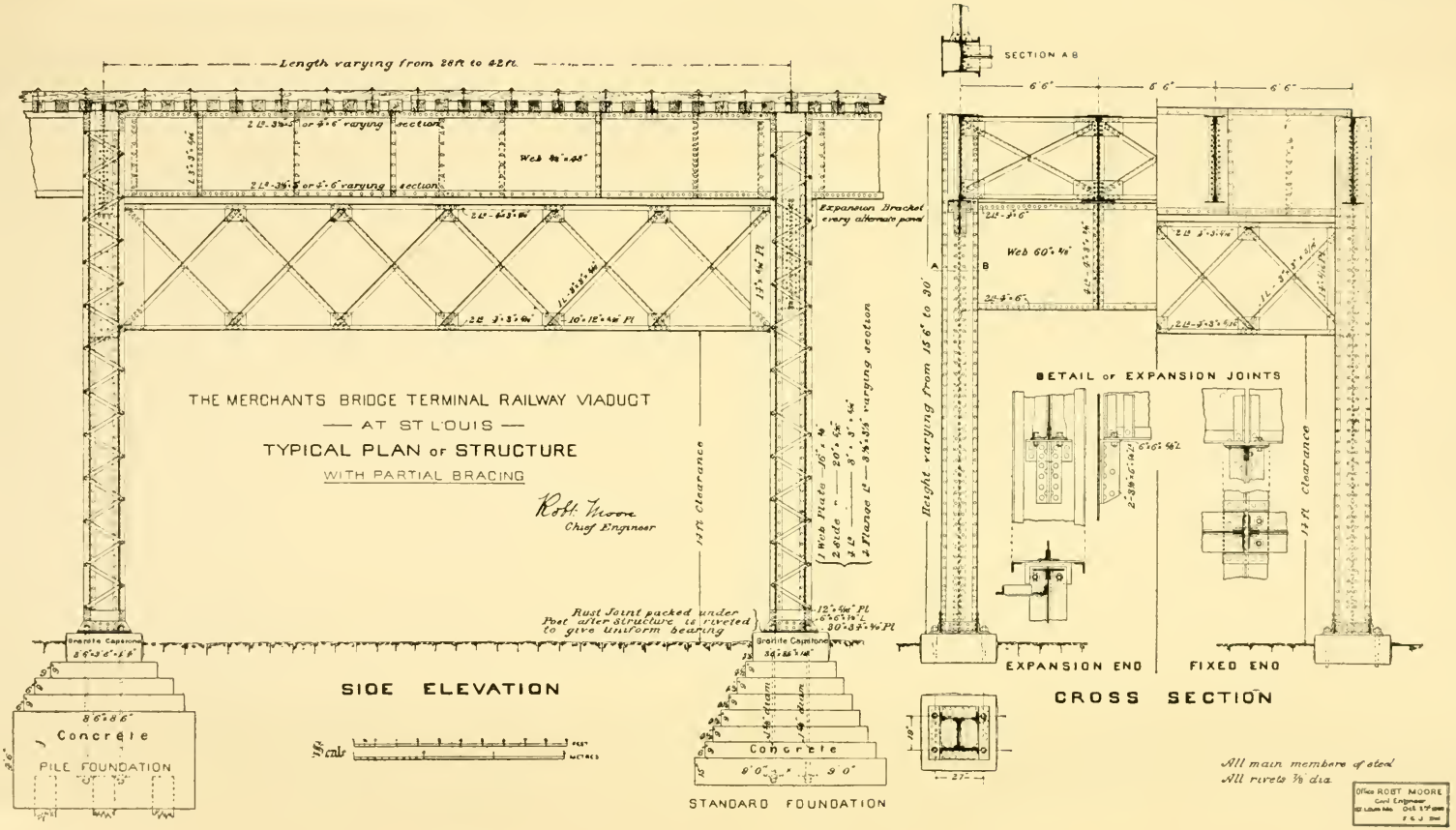
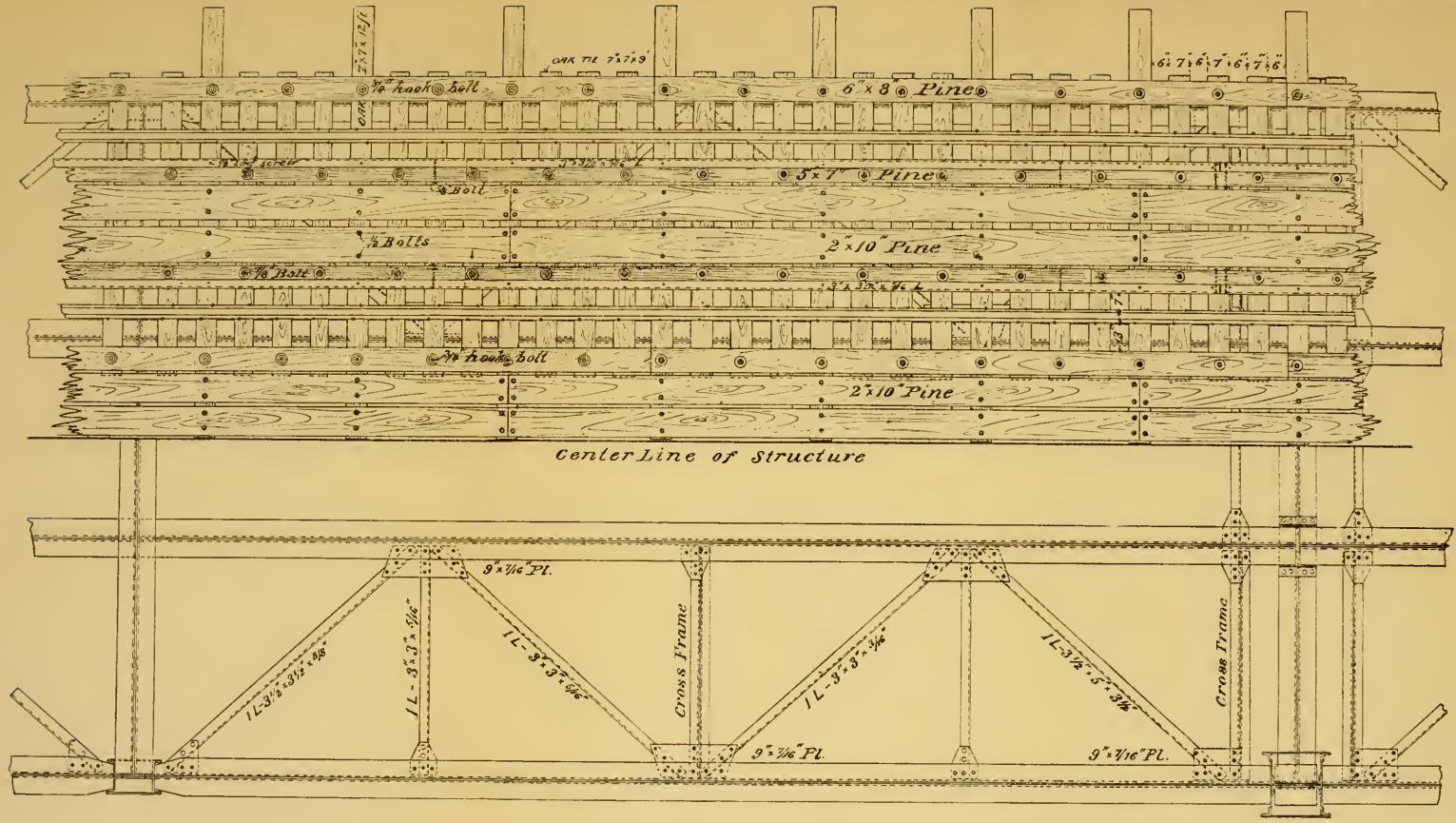


PLATE LXVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 705.
MOORE ON MERCHANTS' BRIDGE TERMINAL RAILWAY



THE MERCHANTS BRIDGE TERMINAL RAILWAY VIADUCT

— AT ST. LOUIS —

FLOOR PLAN AND LATERAL BRACING

SCALE IN FEET

Robt. Moore.
Chief Engineer

Office ROBT. MOORE.
Civil Engineer.
St. Louis Mo. Oct. 2nd 1893
F.G.J. Del.

hereafter, in July, 1893, after three years' service, the cement joint was found to be well distributed and very hard. The result was, in fact, better than would have been attained by efforts to secure perfect workmanship in cutting and setting the stones, or absolute accuracy in the construction of the columns. As often occurs in practice, it was better to provide an adjustment than to waste time in striving for an impossible accuracy in original construction. In doing the work over again, the writer would make this adjustment a part of the original plan.

In making the iron cement it was found important to use only a minimum quantity of sal ammoniac, and to avoid the use of sulphur altogether, thus confirming the experience already gained on the New York elevated lines.* The formula may be stated thus:

Dissolve 1 oz. of sal ammoniac in 1 gall. of water. Take perfectly clean borings of cast iron and mix with a minimum amount of the above solution. Use as soon as it begins to warm up.

The floor is made of 7 x 7-in. white oak ties, spaced 13 ins. between centers and notched over the girders, to which they are further secured by hook bolts passing through the outer guard rails. The inner guard rails are placed 7 ins. clear from the guard side of the rail, or far enough to allow a derailed wheel to drop on the ties. Both inner and outer guard rails are of white pine, but the inner are shod with 3 x 3½-in. angle iron, to protect them from abrasion in case of derailment. One tie in every four was made 3 ft. longer than the intervening ties in order to form a support for a plank footwalk outside of each track and also between them. Only the inner footwalk was actually laid at the outset, but the outer walks with a hand-rail have since been found a matter of necessity for the protection of the men and the convenient operation of the road. This conforms to experience elsewhere, and, were the writer to do it over again, he would make both the inner and outer footwalks with a permanent hand-railing a part of the original construction.

On curves the outer rail was elevated by means of longitudinal wooden strips laid on top of the outer girder in a channel formed by pieces of angle iron riveted to the top flange. This allowed the use of ties of uniform thickness notched in the usual manner, and simplified materially the work of placing them. The rails on curves,

*See Trans. Am. Soc. C. E., Vol. X, p. 124.

in addition to being spiked, were further secured to the ties by Bush interlocking bolts, using three or four pairs to each rail. The results were very good, but the writer would now add a tie plate and would probably use screw spikes of the type in common use in Europe, as being cheaper than the interlocking bolts, and perhaps equally effective, and would discard the ordinary spikes altogether as unfitted for heavy service. In fact, their complete abandonment for any service can only be a matter of time.

Under the head of painting, the specifications required first a coating at the shop with linseed oil, and, second, a painting after erection, with two coats of paint of a quality and color to be approved by the engineer. The paint used was an iron-ore paint brought in the form of paste to the work and then mixed for use with linseed oil. Several ready mixed paints were found upon examination to contain so much benzine that all were rejected. One paint of this class, which was strongly urged by the manufacturers as "an exceptionally good paint for iron work," was found to contain no linseed oil at all. As a basis for future comparison, part of the structure was painted with red lead, and in a few years definite results of permanent value will be obtained.

Since its completion, the structure has borne the test of actual use very satisfactorily. It is very stiff, and, with a single exception, has so far required no alteration or repairs of any kind. The exception referred to was the removal of a column damaged by a collision with a Missouri Pacific freight train August 18th, 1892. This train, composed of loaded box cars, was backed northwardly on a surface track and was derailed by a misplaced switch about 100 ft. south of Post 95A. Though struck with a direct blow sufficient to turn the cars over at right angles to the structure and bring them to rest, the post was not disabled. The angle and side plate which received the blow were sheared off for several inches near the bottom, and the other side plate and angle considerably bent. But the foot of the post was not moved, and the post itself was continued in service for 11 months, or until July, 1893, when it was replaced. Though a somewhat costly experiment to the company owning the cars, to the company owning the structure and to the engineer it gave a proof of the toughness of the metal and the firmness of the anchorage that was very satisfactory.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

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706.

(Vol. XXXI.—May, 1894.)

THE RELATION OF WHEELS TO FROG POINTS AND TO GUARD RAILS.

By ARCHIBALD A. SCHENCK, M. Am. Soc. C. E.

READ APRIL 18TH, 1894.

WITH DISCUSSION.

The short paper presented herewith is a graphical examination, by means of full-size drawings, of the relation of wheels to frog points and to guard rails. It is chiefly the report made by the writer, as Chief Assistant Engineer of the New York Central and Hudson River Railroad Company, in pursuance of instructions from the Chief Engineer. The rail shown in the drawings is the standard 80-lb. rail of that company. The Chief Engineer, Colonel Walter Katté, suggested the presentation of the paper to the Society, as likely to have interest for its members. It will perhaps call attention more urgently to the desirability of joint action on the part of the Society with other associations to secure proper relations between wheel gauges and track superstructure. It is not possible in a general meeting, and with limited time, to present the subject in an exhaustive manner, and this paper is intended merely as a suggestion for action by the Society, only a sufficient number of facts being presented to exhibit to the Society the desirability of some action.

Two sets of maximum and minimum wheel gauges (back to back of wheels) have in past years been adopted by the Master Car-Builders' Association (see Fig. 1). The first set, in 1885 (see Figs. 2 to 5, Plate LXVIII), gave for *A B*, Fig. 1, 4 ft. 5½ ins. as the maximum, and 4 ft. 5¼ ins. as the minimum, allowed. In 1887 a larger maximum, 4 ft. 5¾ ins., and a smaller minimum, 4 ft. 5 ins., were adopted, and have since been maintained, giving the large allowed variation of ¾ in. This action appears to have been taken without concurrent action by any other large body of officials or engineers in charge of track. It leaves many thousand cars in this country running over frogs and track unsuited to this later or 1887 set of wheel gauges.

Some articles have appeared from time to time, designed to examine this subject entirely or chiefly by means of computations. In this paper computations have been laid aside as much as possible, and an attempt has been made to show, by full-size drawings, as stated, just what the state of affairs is, and how much interference takes place in various cases. There are so many uncertainties as to what the exact dimension are in any case, and the possible starting points for computations are so numerous that the number of computations and of figures soon becomes bewildering, and the mind is led away from definite conclusions. The track gauge point as taken on the wheel is found to lie on a curved surface, without any distinctive feature to identify it. The curves in the flange and tread of the wheel and in the top of the rail head prevent the ascertaining by calculation of any definite point of contact from which to calculate further. The wear of wheel and rail add to the uncertainties of computations. The only method even approximately clear of great possibilities of confusion is to place a full-size drawing of the wheel tread on a full-size drawing of the rail, and ascertain results by following as closely as possible what occurs in actual operating.

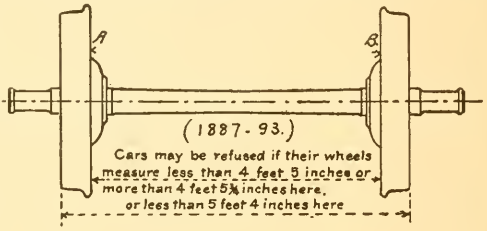
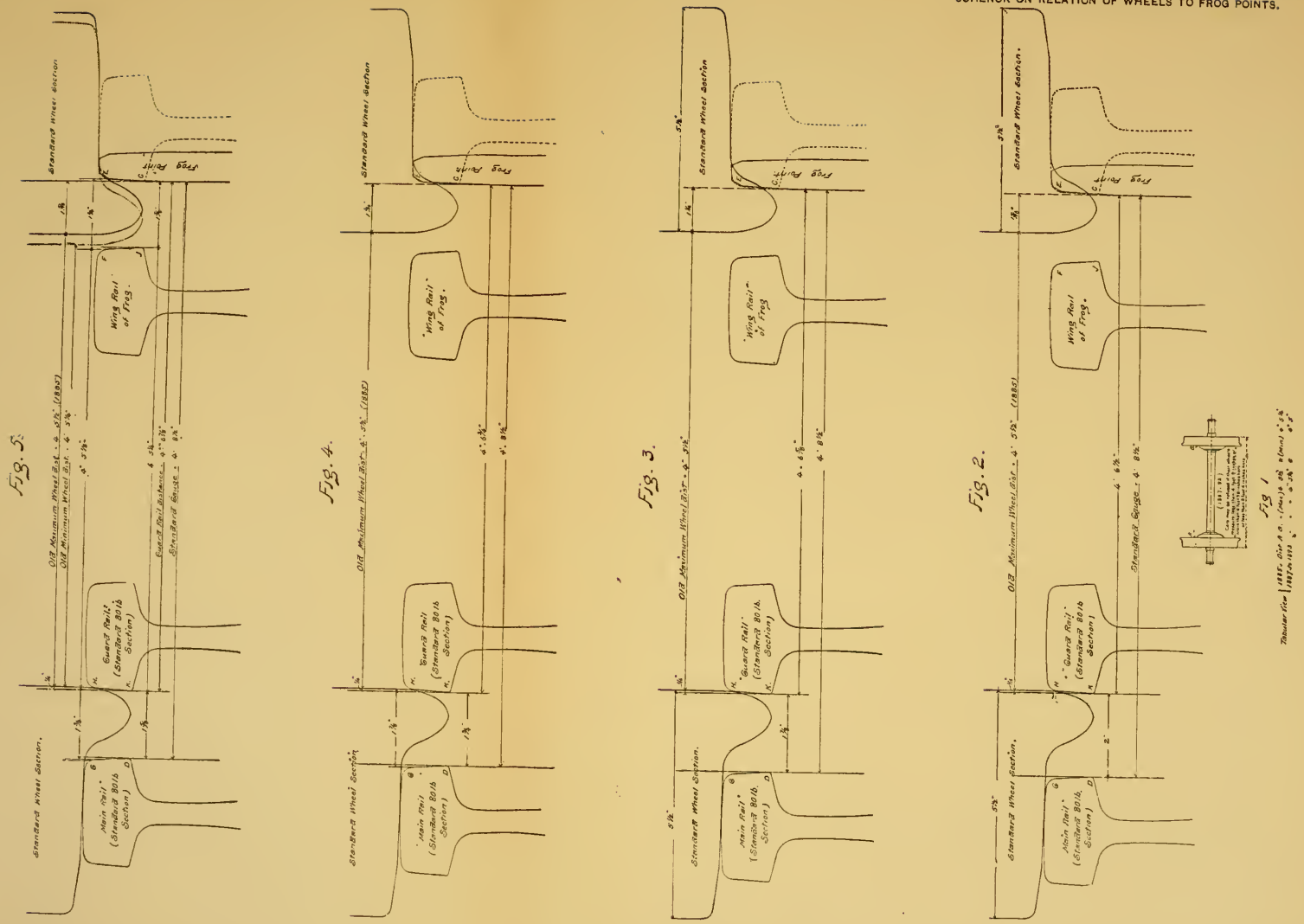


Fig. 1.

Most of the papers that have been written on this subject have

PLATE LXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 706.
SCHENCK ON RELATION OF WHEELS TO FROG POINTS.



aimed to determine a flange thickness, and, by adding twice this thickness to the wheel gauge, to compare the resulting figures with the track gauge. In practice, there is no such definite flange thickness. There is an infinite number of flange thicknesses, because the flange is curved. To speak of "the" flange thickness is as indefinite as to speak of "the" chord of a curve, there being an infinite number. With a sloping rail head, a thickness of flange cannot be taken at the height where the track is gauged (the bottom of rail head in most cases), as the flange there is zero. If we attempt to take a flange thickness at the height of point of contact of flange and rail, this, too, is not definite. The contact is not a point, but a line inclining away from the track and from the vertical, and not having a determinate distance. A graphical solution, made by shifting templates of treads of full size against full-size rail heads, is the only method that even approximates accuracy, and at the same time allows the mind to grasp the whole situation quickly and well, and to make proper comparisons. To attempt to define the situations by figures is like attempting to describe a picture or a scene by figures and dimensions.

The first difficulty met in an examination of this sort is to know just what the gauge of a track is. It seems strange that after so many years of railway experience this should not have been officially defined by any large body of officials. Many consider the gauge as between the upper corners, *E G*, Fig. 2, of rail heads. On one or two roads, and in at least one large manufacturing establishment for frogs and switches, the gauge is taken at mid-height of the rail head. On very few standard drawings is this point definitely determined. In this examination, the gauge of 4 ft. 8½ ins. has been taken as between the points *C D*, Figs. 2 to 5, because the shoulders of the track gauge rod are at right angles to the shaft, and *C* and *D* are the parts of the rail heads in contact with the shoulders, thereby determining the gauge. The flangeways, on the other hand, are often found to be measured with a foot-rule, and improperly between the points *E F* and *G H*, if the rail head be sloping. The proper plan is to measure with a guard rail gauge direct from the side of the frog point to the working side of the guard rail. These are the two points directly concerned in the guard-rail arrangements. The Master Car-Builders' Association formerly had a standard guard-rail gauge (see Fig. 11, Plate LXX). It was of poor design, because too much was attempted in it. It was

arranged, not only to give a distance from the side of the frog point to the working side of the guard rail, but it was also supposed to give the distance from the wing rail of the frog to the guard rail, and from the wing rail to each main rail. At the last meeting of the Association, at Lakewood, it was abolished as a standard, perhaps partly because of the foregoing reason, but also because it was considered that such a gauge was not within the province of that Association. It is properly a matter for the American Society of Civil Engineers, acting in concert with the Roadmasters' Association.

The distance from the side of the frog point to the working side of the guard rail also seems to be indefinite on many standard plans of railway companies. One of the most important roads in this country issues a standard plan with no widths of flangeways stated. The distance from wing rail to guard rail is given at 4 ft. 5 ins. between working sides, but it is not stated whether this is at top or bottom of the rail head. Some of the frogs of that company have a flangeway of 2 ins., leaving $1\frac{1}{2}$ ins. for guard-rail flangeway, with track gauge 4 ft. $8\frac{1}{2}$ ins.

All the examinations, Figs. 2 to 5, are with the 1885 or least objectionable set of wheel gauges. It was considered that whatever was found objectionable for this system would, of course, be more objectionable for the 1887 system ; and that it was necessary to test the latter only upon an arrangement satisfactory for the 1885 gauges, namely upon Fig. 5.

Figs. 2 to 5 are all for a gauge of 4 ft. $8\frac{1}{2}$ ins. with unworn flanges. A worn flange lies further away from the frog point, and does not require consideration in this connection as regards spacing. Various locations of the guard rail with reference to the frog point have been taken, and the interference of a maximum gauge with the frog point is shown in these figures. Any guard rail distance $K\ C$, less than 4 ft. $6\frac{7}{8}$ ins., or guard rail flangeway $K\ D$, greater than $1\frac{5}{8}$ ins., with track gauge as noted, will give an interference at the frog point. A distance somewhat greater than 4 ft. $6\frac{7}{8}$ ins. might be better.

Placing the 1887 wheel gauges on such an arrangement as that shown in Fig. 5, we see in Fig. 6 (Plate LXIX) that the 1887 maximum wheel gauge gives a large interference at the frog point. If we allow for wear of the guard rail, the interference is, of course, increased. Where guard-rail flangeways of 2 ins., or even more, are used, it will

FLATE LXIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 706.
SCHENCK ON RELATION OF WHEELS TO FROG POINTS.

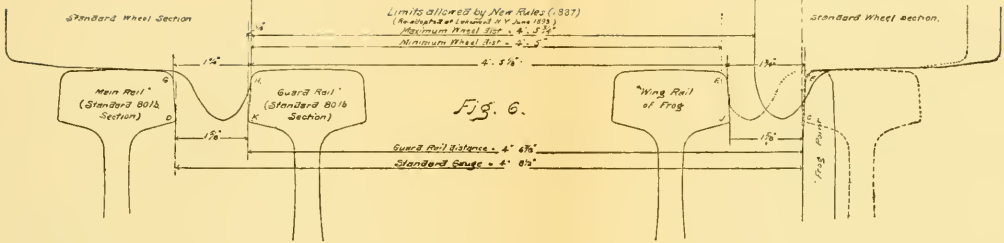


Fig. 6.

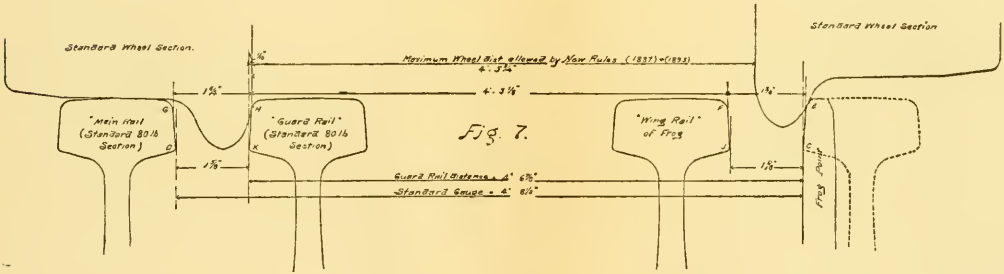


Fig. 7.

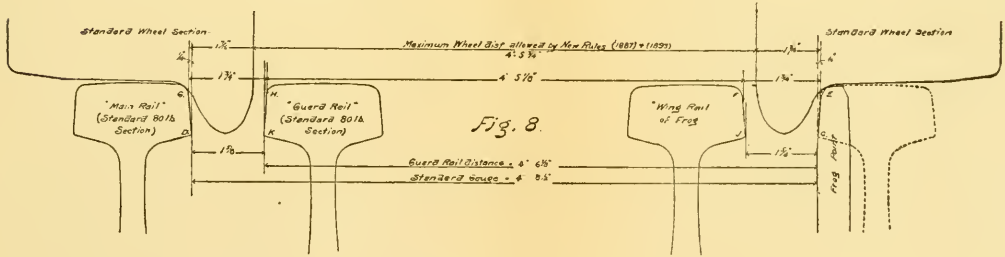


Fig. 8.

Fig 9.

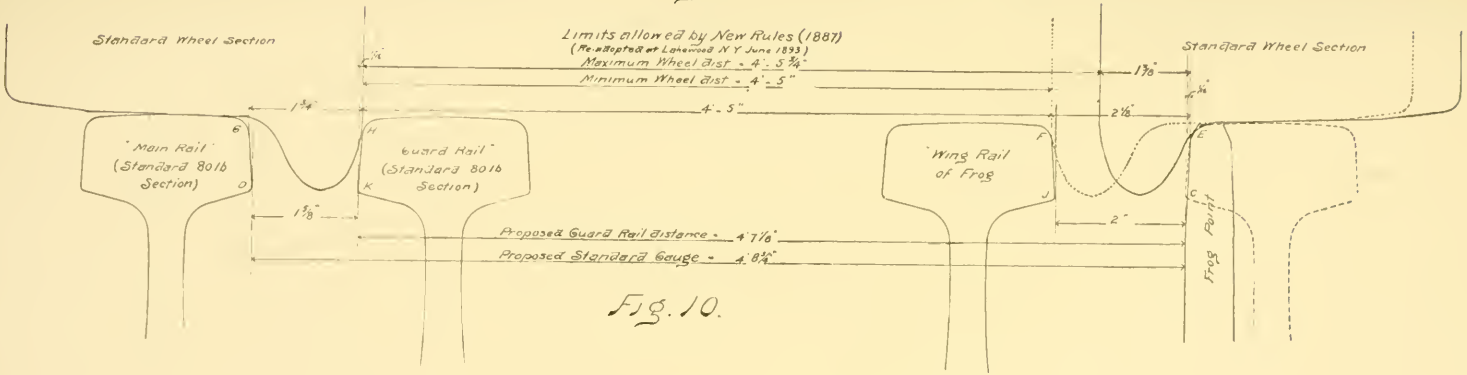


Fig. 10.

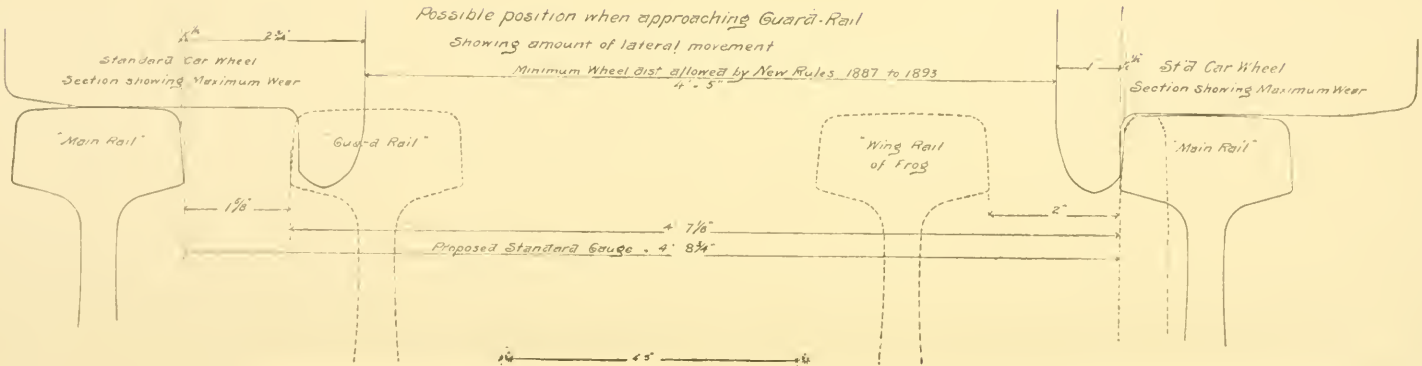


Fig. 11.

PLATE LXX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 706.
SCHENCK ON RELATION OF WHEELS TO FROG POINTS.

be seen that the interference amounts almost to putting the flange on the wrong side of the point. Probably nothing but the impetus of the train carries a wheel of maximum wheel gauge safely past the frog point, in effect there being no guard rail. The 1887 minimum wheel gauge gives some interference with the wing rail. Fig. 7 (Plate LXIX) shows the first movement that probably takes place with the large maximum, running towards the frog point, with the arrangement of rails the same as in Fig. 5. The wheel momentarily rides on its flange and rises, the tread having no bearing. This gives a heavy blow and lateral pressure on the frog point. The second movement is probably that shown in Fig. 8, a shifting of the wheels with a heavy lurch, the guard rail becoming inoperative.

Fig. 10 (Plate LXX) shows how large the lateral movement becomes as the guard rail is approached, with the 1887 minimum and a worn flange. For this reason the track gauge should not be widened more than is necessary in any case. It also gives an additional reason for a return to the 1885 wheel gauges; and for the use of a long taper to the guard rail.

To accommodate the 1887 gauges while they remain in force, both the distance from the frog point to the guard rail, and the width of flangeway in the frog, require enlargement. Fig. 9 (Plate LXX) shows an arrangement that may be fairly satisfactory to those who do not demand a narrow flangeway in the frog. The track gauge is enlarged to 4 ft. 8¾ ins. But the pressure from roads of 4 ft.-8½ in. gauge has of late been so great to reduce flangeways in frogs to 1¾ ins., that the arrangement of 2-in. flangeway in Fig. 9 will not be generally acceptable. There is no alternative but a return to the 1885 wheel gauges.

Although these broad flangeways of 2 ins. are used in many parts of this country (chiefly on roads of 4 ft. 9 ins. or compromise gauge) most of the roads using such a flangeway would probably prefer to reduce it to 1¾ ins. The table on page 514 shows the flangeways in use on many roads. The figures were not obtained direct from these roads in most cases, but from parties making frogs for them, and should be reliable. Where two widths are given, they are as given by two frog companies.

Engineers, roadmasters and frog-builders do not seem to be of one mind as to why frog flangeways should be reduced to 1¾ ins. or less.

Statement of Widths of Flangeways Used in Frogs.

Name of Road or Manufacturing Company.	Flangeways Used in Rigid Frogs.	
	Inches.	
B. C. R. & N. Railway Co.	1⅝	Measured at bottom of rail head.
N. Y. C. & H. R. R R.	1¾	" " "
L. S. & M. S.	1¾	
N. Y. C. & St. L.	1¾	
Baltimore & Ohio	1¾	
" "	2	At bottom of head.
T. & O. C.	1¾	
C. L. & W.	1¾	
C. & W. Indiana R. R.	1¾	At bottom of head.
Northern Pacific R. R.	1¾	" "
Wisconsin Central Co.	1¾	" "
C. M. & St. P. R. R.	1¾	" "
C. St. P. M. & O. R. R.	1¾	" "
M. & St. L. R. R.	1¾	" "
J. & S. E.	1¾	" "
M. K. & T. Ry. Co. Partly	1¾	" "
" " " "	1⅞	" "
T. A. A. & N. M. Ry. Co. "	1¾	" "
" " " "	1⅞	" "
C. & G. T. Ry. Co. "	1¾	" "
" " "	1⅞	" "
St. P. Union Depot Co. "	1¾	" "
" " " "	1⅞	" "
M. St. P. & S. St. M. R. R. "	1¾	" "
" " " "	1⅞	" "
A. T. & S. F. R. R. "	1¾	" "
" " " "	1⅞	" "
St. Paul & Duluth R. R. "	1¾	" "
" " " "	1⅞	" "
Cleveland F. & S. Co.	1¾	For 4 ft.-8½ in. gauge.
" " "	2	For 4 ft.-9 in. gauge.
C. H. V. & T.	1¾	At bottom of head.
" "	2	
Penna. R. R.	2	
Michigan Central.	2	
C. & Northern Pacific R. R.	2	
Wil. & Northern	2	
Cumberland Valley	2	
Western Maryland	2	
W. N. Y. & P.	2	
C. F. & Y. V.	2	
Ohio & Mississippi	2	
Chicago & Erie	2	
C. S. & H.	2	
Phila. & Reading	2	
C. H. & D.	2	At bottom of head.
T. St. L. & K. C.	2	
Pittsburgh & Lake Erie	2	Meas. 5/8 in. below top of rail head.
N. Y. L. E. & W.	2	At bottom of head.
Wheeling & Lake Erie	2	" "
Pan Handle R. R	2	At middle of head.
T. St. L. & K. C.	2	Main track flangeway gauge 4 ft. 8½ ins.
" "	1½	Side track flangeway gauge 4 ft. 8½ ins.
Chicago & North Western	1⅞	At bottom of head.
C. C. C. & St. L.	2⅛	" "
New England Roadmasters' Ass., 1884	1¾	Will reduce to less than 1¾ ins. when gauges become more uniform.
Indianapolis Switch & F. Co.	2	
Ramapo Iron Works	1¾	For 4 ft.-9 in. gauge.
" " "	2	For 4 ft.-9 in. gauge.
Carlisle Manufacturing Co.	2	Meas. at mid height of rail head.
Union Switch & Signal Co.	2	Usual width in all frogs made by them. Measured at ⅜ in. below top of rail head.
Pennsylvania Steel Co.	2	
" " "	1¾	Preferred when wheel and track gauges are reduced.

Note.—In most cases these measurements are taken at bottom of rail head.

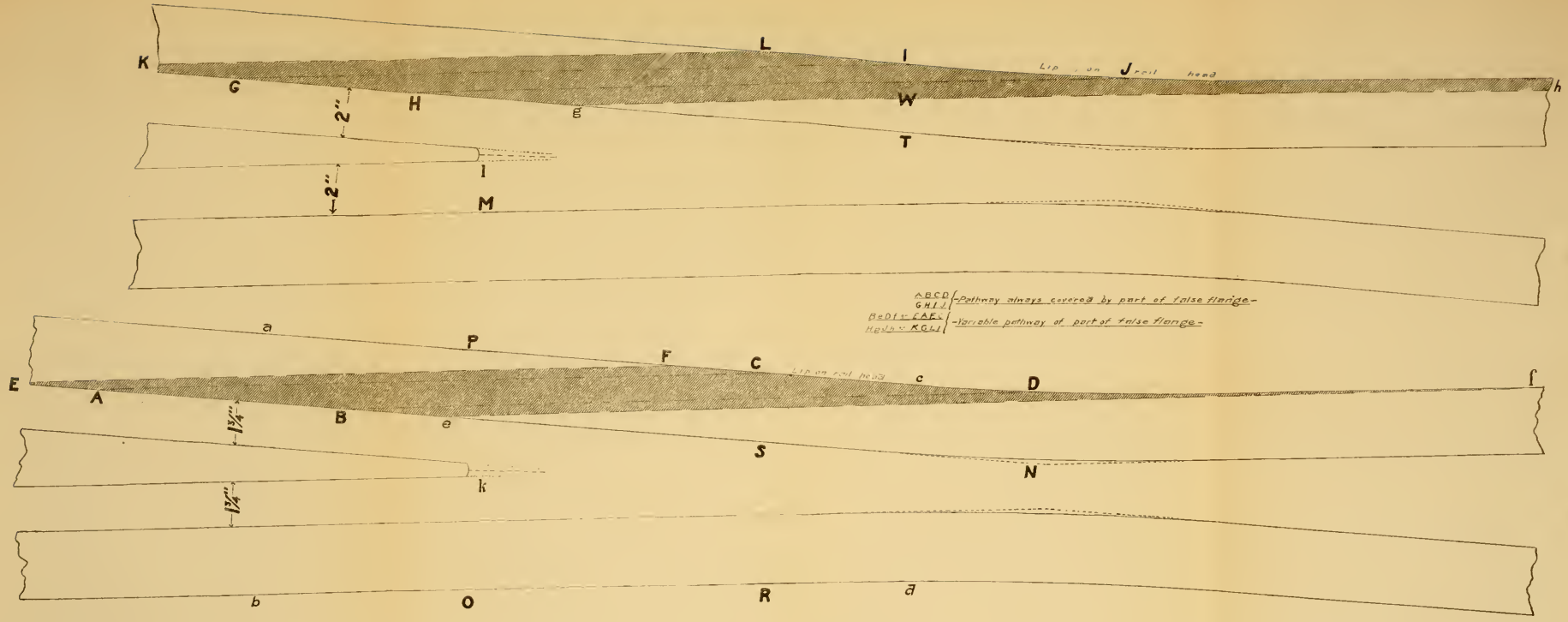

K
L
I
J
Lip on rail head
h
G
H
g
W
T
2"
l
2"
M
ABCD
GHIJ
-Pathway always covered by part of false flange-
BeDf = EAFc
HgJh = KGLi
-Variable pathway of part of false flange-
a
P
F
C
Lip on rail head
c
D
f
E
A
B
e
S
N
1¾"
k
1¾"
b
O
R
d

Fig. 12.

Some of the reasons, as noted in operating or claimed to exist, are as follows:

First, and perhaps the most important.—The rules of the Master Car-Builders' Association allow a loss of $\frac{3}{4}$ in. from rim of tread. This portion constitutes the chief part of the wheel that is to carry the wheel past the throat of the frog, as at *M*, Fig. 12 (Plate LXXI). See also Fig. 16 (Plate LXXIII). The Master Car-Builders appear to have overlooked the value of the outer rim when at the frog point. This reduction, combined with the breakage of a little more when the wheel is at the throat of the frog, lets the wheel down into the throat, causing derailment. Hence the narrower the throat is, the better; and a return to the 1885 wheel gauges is called for. In this connection it would be well to give attention to the filler bearing, which is the secondary bearing in cases of chipped rims.

Second.—With the broad flangeway, the coning of the outer part of the tread allows the wheel to drop deeper into the throat of the frog. With new wheels this causes a blow in running facing point, or a sudden drop to the wing rail bearing, in running trail (see Fig. 16).

Third.—The wear on wing rails is claimed to be greater with the broad flangeway. At first sight this would appear to be the case. An increase of $\frac{1}{4}$ in. in each flangeway, giving apparently $\frac{1}{2}$ in. less bearing of tread on the wing rail, would appear to increase the wear and destruction of the wing rail. In reality there is no gain of increased bearing on the wing rail from the use of narrow flangeways. There is a slight gain in the following respect, namely, that the point where the tread leaves the wing rail is shifted further from the frog point, in a narrow flangeway. With a 2-in. flangeway, the tread leaves the wing rail at *G* (Fig. 12). With $1\frac{3}{4}$-in. flangeway, this point is shifted to *A*, or, in a No. 10 frog, about 5 ins., giving a little wider bearing on the frog point, although the increase is not great, nor as much needed as in the case of a rim bearing. So far as an increase of rim bearing is concerned, it will be noted that the outer part of the tread is crossing the wing rail at an angle. Whatever the flangeway width, there must be a triangular area, *S A C* or *T G I*, where the width of bearing is gradually lessening. With either of the flangeway widths named this point *A* or *G* is far enough from the frog point for the point to furnish a considerable width of bearing. It is therefore doubtful whether narrowness of tread bearing is a large factor in frog

wear. This is still more evident if we consider the effects of false flanges, which is the agency most destructive in frogs. Fig. 12 is intended to illustrate such wear from false flanges. This wear will be found most largely in yards, where the treads of shifting engines are for some reason allowed to have very deep hollows, although running over frogs constantly. The maximum hollow of tire allowed is assumed to be about $\frac{3}{8}$ in. in shifting engines, and about $\frac{3}{16}$ in. in wheels in general service. The false flange simply crosses the wing rail from one side to the other, as indicated by the shaded strips *A B C D* or *G H I J* in Fig. 12. In many frogs in yard service this line will be found either somewhat grooved, if the frog be old; or merely polished, if the frog be new, while much of the area *D E N* or *W G T* is often dull or sometimes almost rusty. The strips *A B C D* or *G H I J* become channeled out soon. These channels or grooves are much deeper at each end than in the central portion of their length. A lip is formed on the rail outside; and would be formed also where the wheel leaves the wing rail inside, but for the wear of flanges against it. A lip is also formed on top of the rail, outside of the line of false flange-bearing. When the areas *D E N* or *W G T* become much higher than the channels cut by the false flanges, these areas are worn down rapidly by new wheels, and in general do not long remain much higher than the false flange grooves. This wear from false flanges is so great in many situations as to be the only one worthy of consideration, and in such cases flangeway widths are not of importance, so far as wear is concerned. Figs. 13, 14 and 15 (Plate LXXII) show in section approximately some of the effects of false flanges. Engine treads widened to $5\frac{3}{4}$ ins. have an injurious action where false flanges of treads of the ordinary $5\frac{1}{2}$ ins. have worn a groove. The outer portion of such grooves being curved, concave upwards, the added $\frac{1}{4}$ in. of tread rests on this curved surface, giving an outward strain on the frog, and probably affecting the axle.

In examining to what extent the narrow flangeway is more desirable than the broad flangeway, many other considerations than mere width of tread bearing should be kept in mind. One consideration is that simultaneous bearing on both frog point and wing rail are seldom secured. Usually either the point alone or the wing rail alone carries all the weight. When both point and tread are new, the bevel on the outer edge of the tread throws all the bearing on the frog point (see Fig.

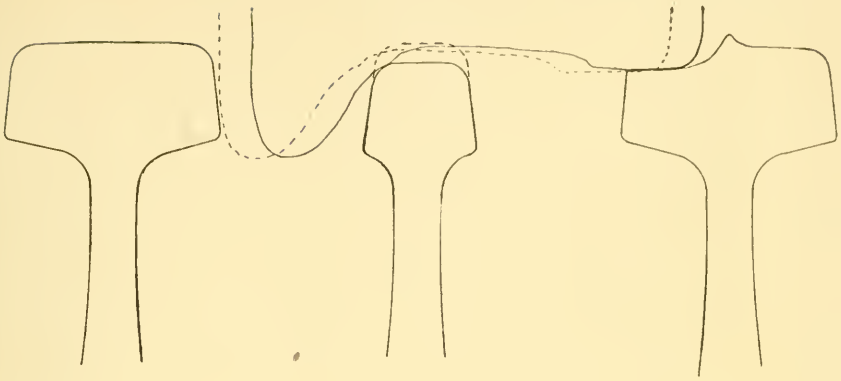


Fig. 15
— Section at a b —

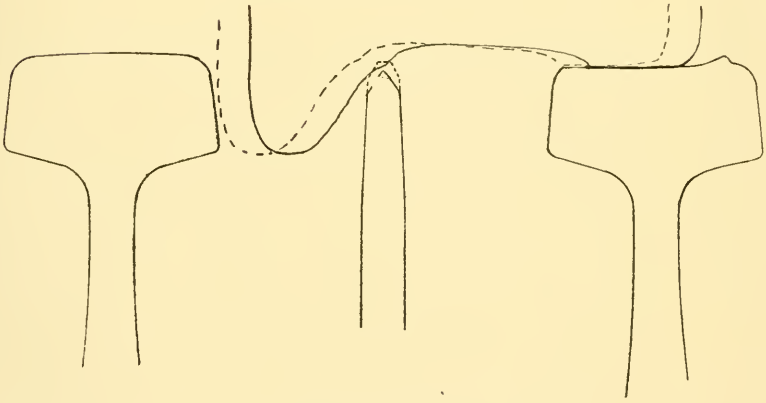


Fig 14
— Section at O P —

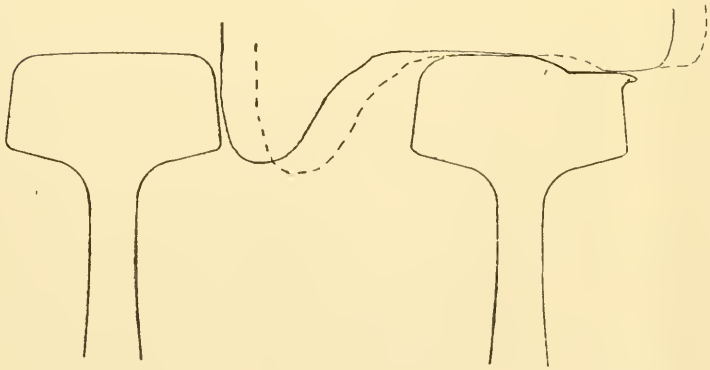


Fig 13
— Section at C D —

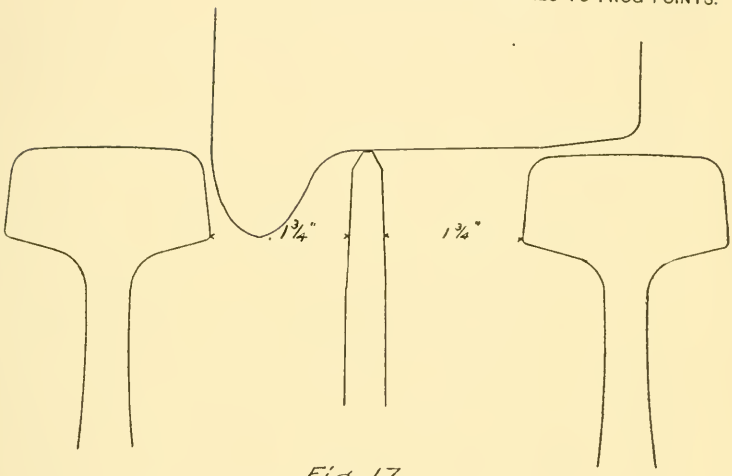


Fig 17
Section at O.P.
New wheel and rail

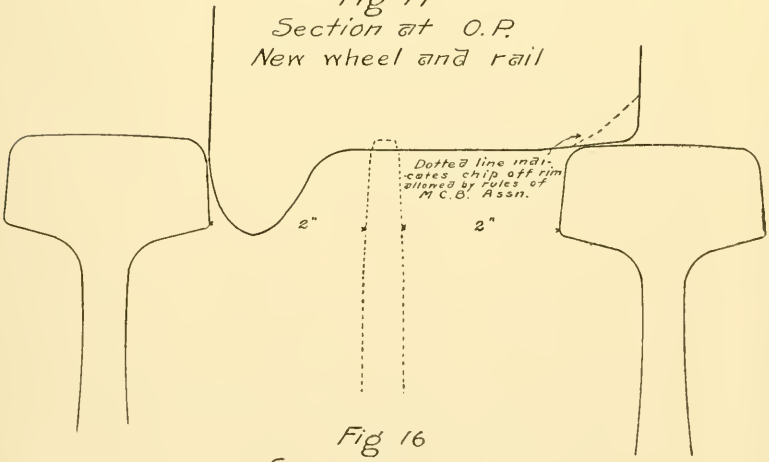


Fig 16
Section at H.M.
New wheel and rail

17, Plate LXIII). A wing rail badly worn by false flanges, or a yielding wing rail, has the same effect. A wing rail bent vertically, part of it low, part "cocked up," also throws all the bearing on the frog point in some cases. In all such cases, the width of flangeway is not of importance. On the other hand, many frog points will be found to "duck" as the wheel approaches them, leaving the wing rail to carry all the weight. The lack of base which the rail forming the frog point has at the point, tends to allow this "ducking." In this case, the width of tread bearing is important, to carry the wheel firmly until the depressed portion of the frog point has been passed. Hollow tires also prevent simultaneous bearing. These failures to secure simultaneous bearing make it desirable to ignore the frog point as a bearing for some distance from the point, and to make narrow flangeways and a return to the 1885 wheel gauges desirable. But so far as wear from broad flangeways is considered, this effect can readily be made too much of.

The destruction of a frog is generally primarily due to shock. This acts both by making the frog too loose and shaky for use, and by indirectly increasing the wear because of such a loose state of the frog. Hence, narrow flangeways will not make a very great increase in the life of a frog unless it is well cared for in other respects. The shock to a frog is generally assumed to be the blow upon the point. It is probable that the lateral blow at *A* (Fig. 12), and also in the angle where the point rails come together, from the false flange of a wheel running trail, strains the frog by a wedge-like action and loosens it more than a blow on the point does. The false flange, before climbing to the height of the rail head, must throw a severe strain on the frog. Such action will take place equally with a 2-in. or a 1¾-in. flangeway.

We may summarize as follows:

1. The necessity of protecting the frog point laterally calls urgently for a return to the 1885 wheel gauges.

2. The dangers from narrow or broken treads call for similar action, and for a modification of the rule about chipped rim of tread.

3. A definite rule about maximum false flange or hollow tire is called for.

4. A return to the 1885 wheel gauges would result in only a moderate gain in durability of frogs, so far as vertical action or wear by wheels is concerned.

5. The appliances used in some spring rail frogs to assist the false flange to climb to the level of the rail head would probably increase the durability of all frogs.

The Master Car-Builders' Association at its last meeting appointed a committee to "prepare maximum gauges for thickness of wheel flanges, and to consider wheel gauges in their relation to the track, and to offer any suggestions as to wheel and track gauges, and to confer with any similar committee from other associations, should such be appointed."

DISCUSSION.

M. J. Becker, Past Pres. Am. Soc. C. E. (by letter).—The "Relation of Wheels to Frog Points and to Guard Rails," which Mr. A. A. Schenck has chosen for the subject of his paper, is a topic which should receive very careful attention and earnest consideration by engineers charged with the construction and maintenance of our railways.

It is but a few years since the gauges of our railway tracks ranged all the way from 5 ft. in the South through the intermediate gauges of 4 ft. 8½ ins. in the West, and 4 ft. 10 ins. in the East, to the 6-ft. broad gauge of the Erie system, compelling, for the interchange of traffic, such makeshifts as double lines of rails, and hoisting machinery for the exchanges of trucks at the connecting points, and broad tread compromise wheels for all rolling stock engaged in through traffic. How the wheels during their periodical changes, and with the always liberal permissible variations between the maximum and minimum gauges, managed to pass through the various flangeways of the switches and frogs seems more of a puzzle to-day than it appeared at the time when this state of affairs existed.

At last the track gauge question is practically settled by the almost universal adoption of the 4 ft.-8½ in. gauge. Only the lines of the Pennsylvania system and a few of its connections still adhere to the 4 ft.-9 in. gauge, which was adopted about 15 years ago as a compromise between the 4 ft.-10 in. and 4 ft.-8½ in. gauges.

The play between the present wheel gauge and the 4 ft.-9 in. track gauge on these lines is 1¼ ins. when rail heads and wheel flanges are new and perfect, and is likely to be more when the rail heads and wheel flanges are reduced by wear. This play causes undue lateral motion and severe shocks at the frog points, and the question of reducing the gauge to the 4 ft.-8½ in. standard is now under consideration.

It was during the investigation of the effect which would be produced by existing rolling stock with its permissible variation of gauge upon the reduced track gauge, with its accompanying frog throats and guard-rail flangeways, that I found the condition shown in the following table:

Case.	Track gauge.	Guard-rail gauge.	Gauge of wheel tread.	Gauge between wheel backs.	Flangeways of frogs and guard rails.	Play between wheel back and guard rail.	Play between track gauge and wheel-tread gauge.	
1.....	4′ 9″	4′ 5″	4′ 7$\frac{3}{4}$″	4′ 5″	2″	None.	1$\frac{1}{4}$″	Close fit.
2.....	4′ 9″	4′ 5″	4′ 8$\frac{1}{4}$″	4′ 5$\frac{3}{4}$″	2″	$\frac{3}{4}$″	$\frac{3}{4}$″	O. K.
3.....	4′ 9″	4′ 5$\frac{1}{2}$″	4′ 7$\frac{3}{4}$″	4′ 5″	1$\frac{3}{4}$″	$\frac{1}{2}$″ less than clearance.	1$\frac{1}{2}$″	Impracticable.
4.....	4′ 9″	4′ 5$\frac{1}{2}$″	4′ 8$\frac{1}{2}$″	4′ 5$\frac{3}{4}$″	1$\frac{3}{4}$″	$\frac{1}{4}$″	$\frac{1}{2}$″	O. K.
5.....	4′ 8$\frac{1}{2}$″	4′ 4$\frac{1}{2}$″	4′ 7$\frac{3}{4}$″	4′ 5″	2″	$\frac{3}{4}$″	$\frac{3}{4}$″	O. K.
6.....	4′ 8$\frac{1}{2}$″	4′ 4$\frac{1}{2}$″	4′ 8$\frac{1}{2}$″	4′ 5$\frac{3}{4}$″	2″	1$\frac{1}{4}$″	None.	Close fit.
7.....	4′ 8$\frac{1}{2}$″	4′ 5″	4′ 7$\frac{3}{4}$″	4′ 5″	1$\frac{3}{4}$″	None.	$\frac{3}{4}$″	Close fit.
8.....	4′ 8$\frac{1}{2}$″	4′ 5″	4′ 8$\frac{1}{2}$″	4′ 5$\frac{3}{4}$″	1$\frac{3}{4}$″	$\frac{3}{4}$″	None.	Close fit.

Note.—The thickness of wheel flange is taken as 1$\frac{3}{5}$ ins.; so that if the gauge between wheel backs is at its minimum of 4 ft. 5 ins., the gauge of wheel treads, which is the contact line between wheel flange and rail head, would be 4 ft. 7$\frac{3}{4}$ ins. If the gauge between wheel backs is spread to 4 ft. 5$\frac{3}{4}$ ins., the gauge of wheel treads would be 4 ft. 8$\frac{1}{2}$ ins.

From the above table it would appear—

First.—That with a track gauge of 4 ft. 9 ins. and flangeways of either 1$\frac{3}{4}$ ins. or 2 ins., wheels with gauge of the extreme permissible limit of 4 ft. 5$\frac{3}{4}$ ins. will pass freely and safely (Cases Nos. 2 and 4 in table above).

Second.—That with a 4 ft.-9 in. track gauge and 2-in. flangeways, wheels with the minimum gauge of 4 ft. 5 ins. will just pass, but rub hard with their backs against the guard rails and wing rails of frogs (Case No. 1 in table above).

Third.—That with a 4 ft.-9 in. track gauge and 1$\frac{3}{4}$-in. flangeways, wheels with minimum gauge of 4 ft. 5 ins. will lack $\frac{1}{2}$ in. for clearance of the guard rails and wing rails of frogs, and will not pass with safety (Case No. 3 in table above).

Fourth.—That with a 4 ft.-8$\frac{1}{2}$ in. track gauge, wheels with gauges of the maximum permissible limit of 4 ft. 5$\frac{3}{4}$ ins. will pass freely through flangeways of either 1$\frac{3}{4}$ ins. or 2 ins., but the wheel flanges will rub very hard against the head of the main rail (Cases Nos. 6 and 8 in table above).

Fifth.—That with a 4 ft.-8$\frac{1}{2}$ in. track gauge and 2-in. flangeways,

wheels with the minimum gauge of 4 ft. 5 ins. will pass freely and safely (Case No. 5 in table above).

Sixth.—That with a 4 ft.-8½ in. track gauge and 1¾-in. flangeways, wheels with the minimum gauge of 4 ft. 5 ins. will just pass, but rub hard with their backs against the guard rails and wing rails of frogs (Case No. 7 in table above).

This shows that with a 4 ft.-8½ in. track gauge, flangeways of 2 ins. width are safer than those of 1¾ ins. width as long as the existing wheel gauges remain in use, and that a judicious widening of the track gauge on curves is necessary.

It also shows that with a 4 ft.-9 in. track gauge the inadvertent purchase and use of a frog with a flangeway of 1¾ ins., of which quite a number are made throughout the country for lines with 4 ft.-8½ in. gauges, would in all probability have caused a wreck, even if the flangeway at the guard rail had been 2 ins.

By the adoption of spring rail frogs, with beveled outside rail attachments for carrying the worn wheel treads over the frog points, some of the difficulties pointed out by Mr. Schenck can be overcome, but I quite agree with him that a determined effort should be made towards the adoption of a standard method for determining the points on the rail heads and frog points at which the track gauge and guard-rail gauge should be applied, and that the question of modifying the permissible variation in the wheel gauge should be considered by the Master Car-Builders' Association in conjunction with the engineers in charge of the tracks.

Henry G. Prout, M. Am. Soc. C. E.—I have not had time to read this paper carefully enough to make any extended discussion of it. I will merely suggest that the question is one which requires co-operation on the part of the people in charge of track work with the people in charge of the rolling stock. It is a fact that there is a very great variation, not recommended by any of the standards, but entirely arbitrary, in the thickness of wheel flanges. Nominally the working thickness of wheel flanges between the point where the wheel comes in contact with the rail head and the back of the wheel is 1¼ ins.; actually it very often reaches 1½ ins. and sometimes 1¾ ins. This variation is enough to cause the wheel to strike against the point of the frog, when if the theoretical gauge—the Master Car-Builders' gauge—was used, it would not do so. The Master Car-Builders' rules of interchange accept a car wheel which enters the minimum gauge, but they have no maximum gauge for the thickness of the wheel flange. The consequence is that one runs into the danger of having the wheel flange so thick that the prescribed standards will actually permit a wheel that will strike the point of the frog.

George R. Hardy, M. Am. Soc. C. E.—The question of the relation of the track gauge and wheel gauge is a very interesting one, and

it brings to mind some very peculiar points. The track gauge undoubtedly is a distance which can be definitely determined. The width usually adopted for the track gauge can be measured between vertical planes, and with rails not formed with a very excessive inclination on the sides of the heads the gauge is very well determined. Different forms of rail head only slightly complicate the exact determination. If a railroad company adopts a form of head with considerable inclination to its sides, then the gauge becomes wider than on a road where the standard rail has vertical sides. It is probable that few realize how these railroads go on operating year after year, with only this little flange of the wheel to keep the track, running at such high speeds, 30 miles or more per hour, and when we consider that there is only about $1\frac{1}{8}$ ins. of wheel flange to protect the whole train service, it is one of the most wonderful things in the world.

I think that we may correctly affirm, notwithstanding the variation in rail heads, that the gauge is accurately defined; but when we come to the rolling stock it is impossible for the Master Car-Builders and superintendents of motive power to determine what that gauge is; it varies according to the relative positions of the wheel and the rail. The problem which was encountered in trying to define an actual gauge for the wheel remains unsolved, and as a temporary expedient or makeshift the mechanics have tried to relieve themselves of the responsibility by prescribing how far apart the backs of the wheel should be. That is much easier because the backs of the wheel can be limited by verticals which may be compared with the distance between the verticals of the rails which measure the gauge, and the difference thus obtained is made up of the slackness plus twice the thickness of the flange. The different positions in which the wheel may be, with reference to the rail, makes a difference in the bearing points of the curves of the wheel and the rail head. If these two curves were alike, the problem might be solved. Four or five years ago the question was in discussion, and at that time it was suggested that a possible actual comparison could be made, if we would make the curve of the rail head and the curve on the wheel that bears against it of the same radius; then, by determining the distances between the centers of each set of two curves, the actual slack of the two gauges could be determined, and also the flange thickness. I do not know that any advance has been made in that direction. It may be that the mathematical determination of what the difference is between the rail gauge and the wheel gauge is unimportant; but in any case the track gauge is defined, while the responsibility of defining a wheel gauge remains unsatisfied. Engineers have tried to determine just how much slack they should give the gauges of the track for curvature. With $\frac{3}{4}$ in. or $\frac{1}{2}$ in. slack in the difference between the two gauges, it seems absurd to think about the question of widening the gauge on curves until we get up to very sharp curves.

H. W. Brinckerhoff, M. Am. Soc. C. E.—I would like to mention a mechanical method of investigating a somewhat similar problem. The Johnson Company, of Johnstown, Pa., made a number of careful experiments to determine the proper sections for guard rails for street railways.

First, an ordinary street guard rail was bent to a given radius; then two wooden discs were made to represent the wheel flanges; these were fastened to a piece of wood so as to keep them in the same plane and the proper distance apart. Then the groove of the guard rail was filled with plaster of paris, and, before it had a chance to set, the wooden discs, connected as above, were slid along the rail, scraping out all the plaster that was in their way and leaving just the shape of the groove that they needed for their free passage and proper guidance. I may add that new guard rails, which promise greater durability and smoothness of running, have been designed in accordance with the result thus found.

It is very obvious that a somewhat similar method could be pursued in investigating the positions taken by a wheel in passing through a frog. If the frog were filled with plaster of paris and the wheel rolled through, it would leave its own record of the positions it had occupied in passing. That would seem to be a convenient way of investigating the question.

L. L. Buck. M. Am. Soc. C. E.—With a larger wheel, it would take more plaster out of the groove of the guard rail. A larger wheel, or shorter radius, or longer wheel base, would each require a wider groove in the guard rail.

A. A. Schenck, M. Am. Soc. C. E.—I wish to allude briefly to one or two items that have been referred to in this discussion. I noticed, however, that the Secretary, in reading one paper, omitted nearly all the figures in it, thus justifying the impression my paper is intended to give, that examinations of this problem by figures is much less satisfactory than a graphical examination.

The information given by Colonel Prout about excess of thickness of wheel flanges has an important bearing on the problem. This excess of thickness, when not in combination with a maximum wheel gauge, is not necessarily a source of danger. The original amount of flange thickness is still left on the right side of the frog point. A flange of ordinary thickness put over a half inch too far upon the frog point might be more dangerous than a thick one, as less flange thickness would be left on the right side of the wheel, to keep the wheel in proper place. But there is an enormous increase of wear by a flange of excessive thickness in combination with the maximum wheel gauge now allowed. What actually occurs with the present maximum wheel gauge, is that the rolling stock is obliged to do on the road what the shopman and the trackmaster should do in advance, and make frogs

and gauges that are suited to the wheels. The wheels plough through the frogs much as a wagon wheel makes its rut in the country road; but by the time the operation is completed there is little left in the frog.

I see that Mr. Becker assumes the flange of wheel as $1\frac{3}{8}$ ins. thick. He also advocates a guard-rail flangeway of 2 ins., and considers the maximum wheel gauge of 4 ft. $5\frac{1}{4}$ ins. admissible. A little blackboard exercise will show:

4 ft. $8\frac{1}{2}$ ins.	4 ft. $5\frac{1}{4}$ ins.
2 "	$1\frac{3}{8}$ "
4 ft. $6\frac{1}{2}$ ins.,	in which to put 4 ft. $7\frac{1}{8}$ ins.

All of the many examinations made of this problem by figures have been based on assumptions of flange thicknesses that are purely theoretical. This flange thickness of $1\frac{3}{8}$ ins. has no definite relation either to the track gauge or to the points of contact and of binding in actual practice. By taking the maximum wheel gauge of 4 ft. $5\frac{1}{4}$ ins. from 4 ft. $8\frac{1}{2}$ ins., we have a balance large enough for two theoretical flange thicknesses of $1\frac{3}{8}$ ins. By an "*argumentum in circulo*" we are then told that the maximum wheel gauge of 4 ft. $5\frac{1}{4}$ ins. plus two wheel flanges of $1\frac{3}{8}$ ins. will go through the gauge of 4 ft. $8\frac{1}{2}$ ins., although it will be a tight fit.

If we would get the normal closest position of a wheel against a rail head, we should shift the one against the other as closely as can be done without lifting the wheel tread off the rail head. Anything closer than this means that we are lifting the rolling stock off the wheel treads and carrying it on the curved flange of wheel. But a determining of a proper clearance of frog point or of a proper wheel gauge by the assumption of any one flange thickness is manifestly impossible, because of the curving outline of this wheel flange, as well as because of the curves in the rail head.

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707.

(Vol. XXXI.—May, 1894.)

THE USE OF CANVAS IN WATER-TIGHT BULKHEADS.

By M. MEIGS, M. Am. Soc. C. E.

READ MAY 2D, 1894.

WITH DISCUSSION.

The use of canvas stretched over loosely fitted boards, to form water-tight constructions, is not new, but is not as well understood by many engineers as it should be. It has a considerable value in engineering work, and, properly appreciated, can be used advantageously in many difficult situations which offer peculiarities requiring special treatment.

The special application which forms the basis of this paper will show one use to which canvas may be put, and possibly may suggest to some of our members experiences in the past or occasions in the future when canvas will solve a difficult problem in a neat and advantageous manner.

The writer considers the use of canvas specially adapted to the following cases:

First.—Construction of water-tight coffer-dams, especially on rock bottoms.

Second.—Suppression of leaks in embankments; as, for instance, the so-called sand-boils.

Third.—Protection of levees and other embankments at critical points during the invasions of high water.

Fourth.—Suppression of leaks in foundations, such as important reservoir dams. For raising sunken vessels, it has been long in use on the western waters of the United States, and probably in other countries.

The special instance referred to, and offered as an example, is the following:

The United States Mississippi River Canal, at Keokuk, Ia., in charge of Major A. Mackenzie, Corps of Engineers, U. S. A., M. Am. Soc. C. E., and locally administered by the writer, has been in constant operation since 1877.

There are three locks, one at each end of the canal, and one about midway. The plan under which the canal was built provides no guard gates at either end, as is the general and very commendable practice at present. The two upper locks have been drained from time to time, and inspections and necessary cleanings and repairs made, as occasion demanded. The upper lock was on two occasions separated from the river by an ordinary coffer-dam of plank and mud, no gravel being at hand or easily gotten, and a dredge being available, which could furnish quickly any quantity of rather stiff and heavy silt or gumbo of the usual Mississippi River variety.

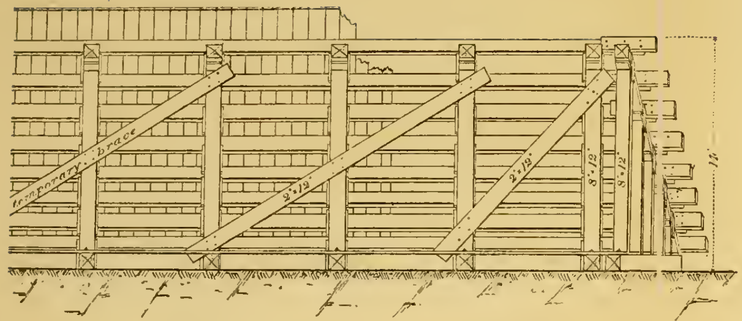
On each of these occasions the work had to be carried on after the close of navigation, under great difficulties from inclement weather, and on one of them the imperfectly consolidated dam, in a partly finished condition, upset just as a Dakota blizzard made its appearance and froze up towboat, dredge and four barges of mud, putting a stop to the work for the rest of that winter. This disagreeable experience was so well remembered by all connected with that episode that it was agreed that almost any other form of coffer-dam than a mud one was preferable for use after the close of navigation on the Des Moines Rapids Canal.

At the close of navigation, in 1893, November 15th, the lower lock had not been inspected for 16 years, and there was evidence in the working of the submerged machinery connected with it that the time had come when the lock must be drained, and examinations and proper repairs executed.

PLATE LXXIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 707.
MEIGS ON CANVAS FOR BULKHEADS.

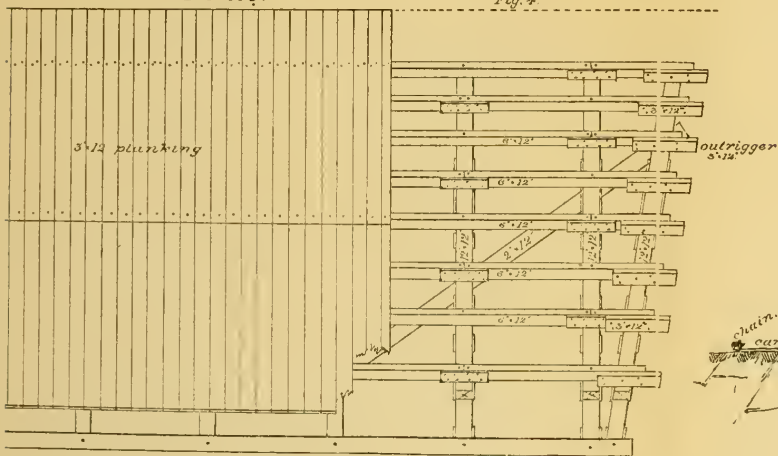
U.S.D.M.R.Canal.
Canvas and Plank Bulkhead
at Lower Lock.
Keokuk November. 1893.

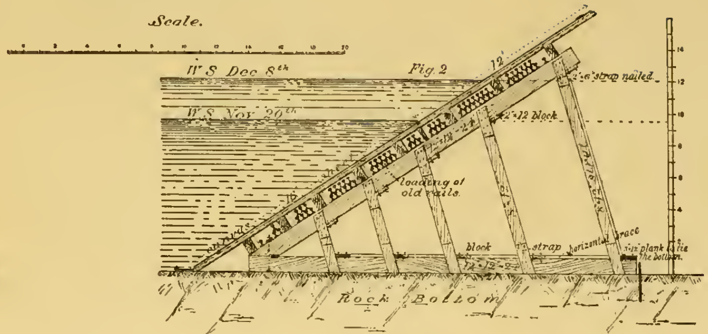
Rear Elevation.



Plan.

Fig. 4.





Section 1.

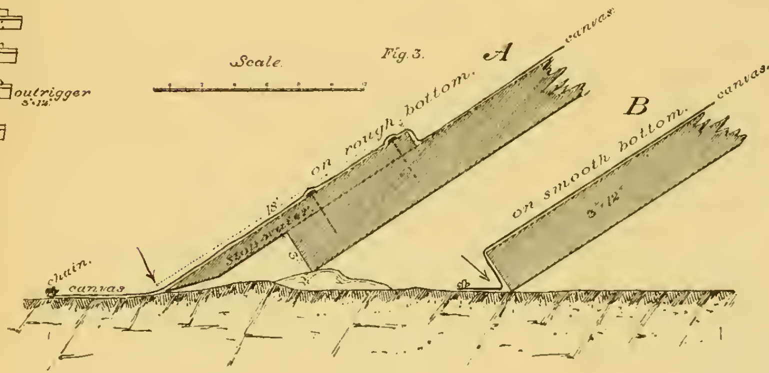
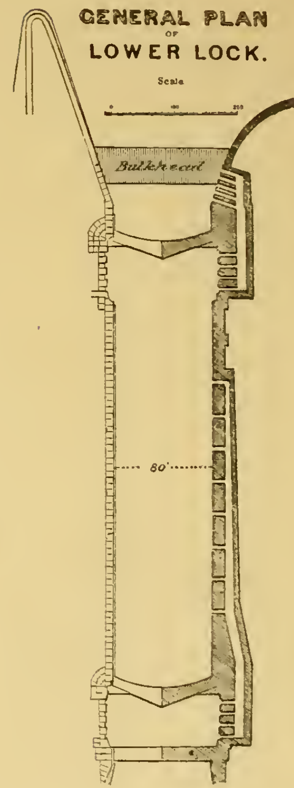


Fig 1



The filling and emptying culverts, 8 ft. wide and high, and the 16 iron culvert gates, 3 ft. wide and 6 ft. high, had to be examined, and the working of the latter showed that the slides on which they moved had to be replaced.

The leakage of the gates, too, had become serious, and their condition was a matter of speculation so far as the lower 9 or 10 ft. was concerned.

These gates are 46 ft. wide, each leaf, and about 27 ft. high, weighing some 40 tons each.

All these repairs and examinations were quite beyond the scope of a diver, and there was nothing to do but to drain the lock.

The canal being in constant use during the navigable season, the repairs had to be made in winter weather, which in these latitudes often sets in in the latter part of November.

The river is usually at a low stage in the latter part of November, but, as soon as the cold weather comes, the anchor ice causes sudden fluctuations in the level, generally rising to about a 4-ft. stage, until the river closes, when it may rise considerably farther, depending on the accidents of gorging below or above the foot of the canal. Hence a coffer-dam of less than 16 ft. high above the bottom of the river, or 8 ft. above low-water line, was not deemed safe. To have built this of gravel and timber in freezing weather was a job to be contemplated with little pleasure.

The writer had at his disposal a 50-H. P. suction dredge, with 14-in. suction, and a rotary Van Wie pump, also plenty of 12-in. discharge pipe, mounted on pontoons, and concluded to use this pump in preference to an engine on the lock wall, and an ordinary drainage pump, which would, however, have answered equally well, but was not on hand.

It was proposed to drain the lock with this dredge, allowing the boat to settle in the mud at the bottom of the lock as the water left it, and to complete the drainage with a 3-in. discharge pulsometer. The lock, being 350 ft. long and 80 ft. wide, a flat place on the bottom was selected and the dredge placed over it, the necessary length of discharge pipe placed in position on its pontoons, and this arrangement left nothing to be desired in the way of pumping machinery, as the result shows.

The point selected for the bulkhead and shown clearly in Fig. 1,

PLATE LXXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 707.
MEIGS ON CANVAS FOR BULKHEADS.

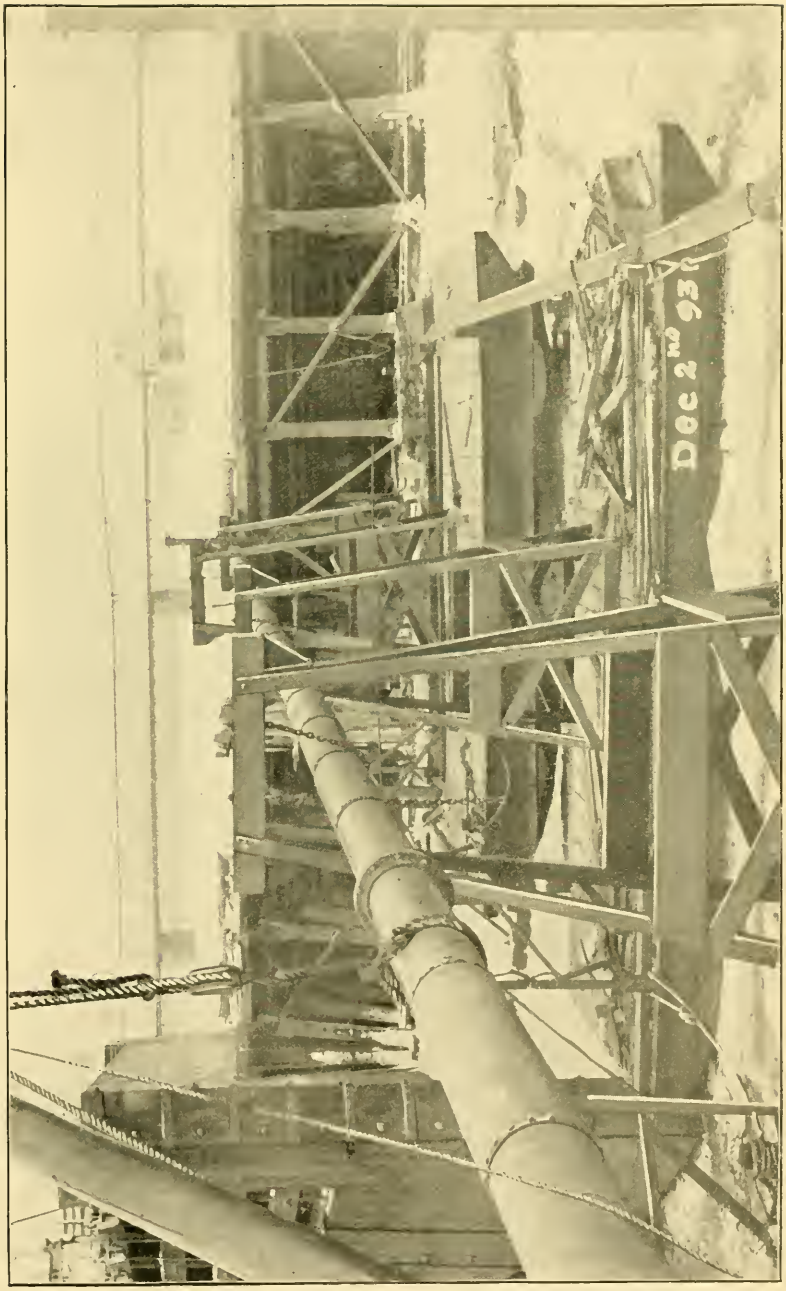


Plate LXXIV, was some distance (about 40 ft.) below the lower mitre-sill, where there was a smooth rock bottom, the ends of the dam abutting against the flaring ashlar wing walls of the lock approach.

The bulkhead consisted of thirteen triangular bents (Fig. 2, Plate LXXIV) of 12 × 12-in. timber for the rafter (or cap) and sill, and 8 × 12 ins. for the struts. The bents were spaced 8 ft. apart, except the two end bents, which were canted so as to loosely fit the angular plan of the wing walls, and were made less than 8 ft. from the next bent. The slope of the upper face of the dam was made 1½ to 1 in order to secure more downward pressure than horizontal thrust, and thus prevent sliding. An additional precaution was also taken to prevent sliding by setting three pins of 1½-in. iron behind three of the sills about the center of the lock. The holes for these were drilled after the frame had been weighted, and rested on the bottom, and were bored through a 2-in. pipe with hand drills from a barge moved to the upper side of bulkhead. The result showed these precautions might have been omitted, as the framework never slid at all.

The rafters or caps supported purlines of 6 × 12-in. timber, shown in the accompanying plates, and on these, when the framework was settled in its place on the bottom, the apron of 3 × 12-in. plank was spiked.

The original plan was to use purlines 16 ft. long and butt them on the caps, as shown in Plate LXIV, Fig. 4, but as 18-ft. timber was of much greater use in our barge repairs, it was concluded to use 18-ft. timbers and let them overlap, as Plate LXXV clearly shows. A single ⅝-in. screw bolt secured the purlines to each rafter, the hole being bored a little large, to facilitate the dismemberment of the structure when it had served its purpose. The struts were fastened to the rafter and sill with 2-in. plank nailed to each.

No "gaining in" was used, the idea being to cut the timber as little as possible, in order to keep it for other purposes. At the lower end of the rafter a 1-in. bolt was passed through it and the sill, and a key of oak wood driven in, to prevent the rafters sliding on the sill, owing to the resultant of the normal water pressure and the angular position of the struts. The structure was lightly braced diagonally, both in plan and on its vertical members, to prevent deformation while being placed in position.

The plan, Plate LXXIV, shows these of 2 x 12-in. plank. Plate

LXXV shows much lighter bracing, being about 2 x 6-in. scraps picked up in the boat yard. It was designed that the structure should have considerable flexibility, and the purlines were kept short for this purpose. The vertical diagonal braces were made light, so that they should give way if necessary and allow the bents to adapt themselves to possible inequalities of the bottom.

Careful soundings showed little variation in the bottom however, and when the water had been removed the bents were found to rest on a smooth concrete floor, kept perfectly free of all deposit by the powerful scour from the discharge culverts.

The thirteen bents and the purlines were set up in the United States Dry Dock, 2½ miles up the canal, and floated down to place by means of a tow boat and two flat boats, suspended between the latter by six ⅝-in. chains. The frame, in getting out of the dock, had to pass over 4½ ft. depth of water, owing to the low stage of the river, and the strain on the high side of the structure carried by three chains was considerable.

Fig. 2, Plate LXXVI, shows the device used to suspend it.

No difficulty, however, marked its journey from the dock to the site of its final location; it was a clumsy thing to tow, and it was urged along gently, to prevent any distortion *en route*. Arrived at the proper place, it was floated over the marks made beforehand, and readily sunk with old rails. The chains enabled it to be controlled as they were slacked off from the barges, and, in case of an error, to be raised and moved slightly, should it be necessary.

About 60 000 pounds of old rails were borrowed from a friendly railroad, and used in sinking and ballasting the structure. The spaces between the purlines were admirably adapted to receiving and holding the rails, and the rails were skidded into place from the lower barge, one at a time, the sinking being kept uniform throughout. A portion only of the rails were required to sink the frame; the rest were added to overcome the buoyancy of the 3-in. covering presently to be put on. The load put on was solely to overcome buoyancy. None was used to give the structure further weight, the water being calculated to hold it down sufficiently. Up to this point the work had been carried on without the services of the diver, who arrived in time to make an examination of the bottom and see that no rough points or boulders interfered with the proper contact of sills and floor.

Fig 1.
Fig 2.
20 - 30'
Fig 3
Fig 4
Fig 5.

During this stage of the work the depth of the water was between 9 and 10 ft. All the ballast of rails having been put on, the first tier of sheathing plank, 3 x 12 ins. and 16 ft. long, was added rapidly. The diver, stationed on the bottom, slid each plank close against its neighbor and saw that it touched bottom properly.

Two spikes were used to secure it to the lower purline, and one spike was used at the upper end. The spikes below water were started before sending the plank down, and driven with a "shot gun" from the barge above (Plate LXXVI, Fig. 4). The shot gun consisted of a ¾-in. pipe with an iron rod in it for a hammer. All the diver had to do was to place the shot gun on the spike and give the signal by rapping on the pipe, when two or three strokes of the rammer sent it home. The upper end of the plank was out of water generally, and the spike in that end was driven by an ordinary hammer.

We found it very important that this upper end should be out of water, and wished this first tier had been 18 instead of 16 ft. long for that reason. In a few places the spike had to be driven when the upper end was 3 or 4 ins. beneath the surface, and disagreeable splashing resulted. The second tier of plank was made 8 ft. long and placed as the lower tier progressed. Had the water required it, a tier of short 1-in. plank would have been used on the upper purline, and a narrow strip of canvas laid over it; but this was held in reserve and was not, as it happened, required.

The canvas sheet, which was designed to give tightness to the apron, was of two breadths of 10 ft., and one breadth of 6 ft., wide, sewed together edge to edge for convenience, and about 4 ft. longer than the extreme length of the apron. Some old ½-in. and ⅝-in. chain was sewed to one edge continuously to act as a sinker and insure the lower edge of the canvas sheet hugging the bottom tightly. A few stones laid on it would have answered the same purpose, but not so well. The canvas was 12-oz. duck.

The sheet was spread under water by the diver. It lapped on the bottom about 12 ins., covered the face of the apron and extended some inches up the face of the wing walls at the end of the dam. Cleats were nailed on the angle between the apron and the wing walls; these were of 1 x 4-in. strips, nailed with 2-in. wire nails about 12 ins. apart. The upper edge of the canvas was also lightly cleated to the planking in a similar manner. No other nails were driven in the canvas, which was designed to be cut up into tarpaulins eventually.

Where the plank touched bottom no beveling of the end was done. It was intended originally to use "stop waters," as at *A*, Fig. 3, Plate LXXIV, of 1 x 12 ins., 18 ins. long, to make a close joint. In one place, where there was a ragged hole in the rock bottom, these beveled stop waters were used, and, had the bottom been less regular, would have been applied all along the junction of the apron with the bottom. The diver thought them unnecessary, except in the one place mentioned, and the result showed he was right.

All being in readiness, the pump was started and the immediate lowering of the water behind the dam showed that the canvas had taken hold at once. About 3 ft. of water were taken out before night, and the pumping operations suspended till next day, in order to avoid accidents. About six hours' pumping in all laid the lock chambers dry. The placing of the dam in position began November 20th, and on the 25th it was pumped dry. The diver was employed five days in all. Some of these days it was so cold he could not work continuously and had to come up, take the suit off and warm himself. The weather was cold and freezing. The river closed two days after the lock was pumped out.

The tightness of the dam astonished everybody. Large quantities of fish, the writer regrets to say, found themselves deprived of their natural element, and were carted off by the barrelful. The small fry were a serious nuisance by clogging up the pulsometer.

The length of the dam was 96 ft. on the lower edge of the apron, and about 8 ft. less at the back. No drip was seen on the under side of the apron, nor could any water be discovered coming under the bulkhead where it joined the bottom. All leakage could have easily been pumped by hand with one hand pump, and even that came through open joints in the masonry around the ends of the bulkhead and not under the canvas. A pulsometer, with 3-in. discharge pipe, worked intermittently, kept the lock chamber dry. Some of these leaks, which at first seemed serious, were cured by dumping rotten stable manure in their vicinity, a load of manure having been provided beforehand for this purpose.

One could walk under the structure anywhere without getting wet.

At the beginning of operations and while the sheeting was being put on, the depth of water varied between 9 and 10 ft.

The dam remained in position from November 25th to December

8th, when, repairs being completed, the water was admitted and the dam removed. The sheeting was easily taken off after the removal of the canvas, which latter was cut up and converted into tarpaulins. The rails were hooked up, one end at a time, and hoisted out by means of a steam winch and a gin pole on the lock wall.

Not a rail was lost. The wrecking of the rest of the structure was easy, and all of the timber was saved for other purposes. The spikes used were ¼-in. wire nails, 6 ins. long, and were easily pulled out.

The whole work was in marked contrast to our mud dam experiences, where the dam was turned over after use to the tender mercies of a dredge, that made kindling wood of all timber. The entrance to the lock was also left perfectly clean.

Some anxiety was felt as to the effect of the ice on the canvas. It was thought it might freeze to the canvas, and, as the river raised or fell, tear it off, but no such an effect was seen. It froze tightly to the plank beneath and took care of itself very well.

The greatest head of water that the bulkhead withstood was about 12 ft. This was after the river had closed. There was no indication that the canvas would not have withstood 25 ft. of head just as well. The pressure of the water forced the canvas into all depressions and cracks and increased the tightness with the pressure.

An apron of poles would probably have answered every purpose to support the canvas. The tendency of the water to force it into all openings overcomes all but the grossest defects of workmanship.

The structure described above was on a smooth rock bottom, but it would probably work equally well on gravel, care being taken to give a sufficient surface of canvas, say 4 or 5 ft. on the bottom, to get the required resistance to the infiltration of water.

The bulkhead at the lower lock, with its 12-ft. head of water on it, showed that, with that amount of head, the seepage was small. How much greater head 12-oz. duck can withstand without allowing much water to pass through is not generally known, the writer thinks. The following experiment was tried to determine this point.

A piece of 4-in. steam pipe was fitted at one end with a pair of common flange coupling plates, between which a gasket of 12-oz. duck was clamped, closing that end of the pipe. The other end was reduced, and connected with a small force pump, and a pressure

gauge. The figures below show what happened when the pressure was applied :

Pressure, 1 lb., canvas shows no signs of any water coming through.

" 2 lbs., water comes through the pores in rapid drops, so that rapid pumping is required to keep up pressure.

" 5 lbs., water comes through so fast that the pump maintains this pressure with the greatest difficulty.

Supposing that mud might have something to do with making it tight, for no leakage at all was observable in the dam, already described, due to seepage through the canvas, the 4-in. pipe was opened and a couple of handfuls of mud placed therein, and shaken up with the water. Pressure was again applied with the following result :

Pressure, 2½ lbs., canvas perfectly water tight.

" 5 lbs., slight drip in a few places, but water comes through clear.

" 7½ lbs., same as above, a little more leakage.

" 10 lbs., still more seepage, but not large.

" 50 lbs., water still comes through clear.

Leakage through canvas about as in first experiment with 10-lb. pressure.

The canvas was still unruptured, though supporting a pressure on a circle of 4½ ins. diameter, in the neighborhood of 800 lbs., without tearing.

This experiment goes to show that muddy water is favorable to the tightness by closing up the pores of the cloth.

In very clear water either heavier canvas or treatment of the lighter grades with some filler, like linseed oil, would be necessary to prevent the leakage from assuming troublesome proportions in a dam or bulkhead depending on canvas under heavy pressure.

In removing the canvas from the lower lock bulkhead, the side toward the water was found to be covered with ¼ to ½ in. of mud, though the water was generally pretty clear while it was submerged.

It had evidently filtered this out of the water. The side toward the plank was perfectly clean, the mud being all collected on the other side.

This action of canvas is made use of in well-known filters.

The writer has assisted at the raising of several sunken steamers, when canvas played the most important part. The diver's first care is to close the break in the hull by nailing planks across, and over this a patch of canvas of proper size, fastened to the hull outside the plank with laths, using $1\frac{1}{2}$-in. wire nails.

Then a bulkhead of vertical plank is built around the boat like a fence, the upper ends projecting above water, and the lower ends spiked to the edge of the hull. No particular care need be taken to make this planking tight; rough boards are used just as they come. The sheet of canvas, wide enough to lap over the lower ends of the boards and be battened to the hull 4 or 5 ins. from the ends of the boards and to reach above water, is usually in one width, made by sewing the necessary breadths together. It could be in several widths lapping on each other and secured with laths nailed over the joints; but this greatly increases the work of the diver.

The writer has seen a coal barge (Plate LXXVI, Fig. 3) 200 ft. long, 25 ft. wide, loaded with coal, raised in this manner.

The barge began to float so soon as 4 ft. of water was taken from the inside of the bulkhead, so that the head of water on the bulkhead was never very great.

On another occasion the writer witnessed the raising of a large Mississippi River steamer 240 ft. long and 26 ft. wide (Plate LXXVI, Fig. 1). This boat was pretty heavily loaded and had a hole 2 x 4 ft. made in her bow by a snag, and sank in a few minutes. The bulkhead planks in this case were nailed on her guards, 3 ft. beyond the hull. The canvas was secured to the hull 12 to 18 ins. below the line of her gunwale. The under surface of her guards was naturally very uneven, consisting of 3 x 6-in. outriggers or floor beams, on top of which the deck plank rested, but the canvas found its way into all these uneven places and made all tight. This boat began to raise when but 18 ins. of water was removed from the inside of the bulkhead so that the pressure was light. Had it increased to 5 or 6 ft., the water would probably have torn her guards off by lifting them up.

It sometimes happens that a boat settles down in soft mud. When this is the case, a much greater lifting power is required to tear it from the bottom, which attaches itself to the boat like an enormous sucker.

In one case the boat had sunk so there was 8 ft. of water on the

main deck, and this deck was laid bare before it would leave the bottom. It is needless to remark that when the bottom did let go, the boat bounced up to the surface with a quickness that threw every one on board off their feet.

Where the boat settles down on sand no such difficulty occurs. It would, of course, on a mud bottom facilitate operations to direct a stream from a hose under the bottom so as to break the connection with the boat, and allow the water to get in.

Plate LXXVI, Fig. 2, shows the usual method employed in raising a sunken structure (in this case a boat) by means of barges and chains. It is not new, but is simple, operates perfectly, and is useful to know. The sketch shows pretty well how it is worked.

a a is a stout lever of wood. In steamboat raising on the Mississippi by chains, usually an oak or elm tree is used, of 12 ins. at the butt, and as long as the width of the barge will permit. *b b* are two U-shaped toggles, of square iron, of such a width as to slip easily on the chain, but too narrow to allow the chain to pull through. Depressing the lever draws the chain up through the hole in timber *c c*, and the toggle is then slipped on the chain one or two links lower down.

Raising the lever, the chain above the lower toggle is slacked, and the upper toggle is moved one link lower. Tremendous strains can be applied to the chains in this way, sufficient to break the chain or lift the load. This method of raising boats is employed when no diver is available and the boat is small. Also in deep water it is used when a bulkhead with its top out of water is impracticable.

It is hard on the boat and most laborious, and is seldom employed on the Mississippi on a boat of more than 150 tons.

It only requires some chains and toggles, which can be made anywhere, the rest of the tools being taken from the woods near by.

Plate LXXVI, Fig. 3, shows the coal barge in process of raising. In Fig, 3, *b b* is the bulkhead plank represented at one end without the canvas *c c*, which is shown, being unrolled along the side. *f f* is a frame to nail the upper ends of bulkhead to. Fig. 1 represents the steamboat raising, where the bulkhead was secured to the guards at *a*. Fig. 1 (*b*) shows the position of the bulkhead when the guards are considered too weak to stand the strain of the other method. It is much more troublesome. Usually freight has to be got out of the way, and a cleat nailed on the deck to foot the plank against.

It should be remembered that the canvas skin in all these operations is what gives the element of water tightness. All the rest is only a frame on which to stretch it.

Of course a large hole might allow the canvas to bag in it so as to be pulled bodily through and rupture the canvas, or so disarrange it as to cause a leak somewhere else.

The ideal is a basket with a skin of canvas outside. The water finds every leak, and if the canvas covers the leak, even, if a very small one, it pushes the cloth in and stops the hole.

Plate LXXVI, Fig. 4, is a sketch of the "shot gun" mentioned before. It is invaluable in such cases as the bulkhead at the lower lock. There is nothing new about it, but it is given, as it may have novelty for some people.

Plate LXXVI, Fig. 5, is a suggestion for stopping springs in foundations that are to be water tight. *c c* is a sheet of canvas of proper size, in the center of which a patch of cloth is sewed on, a sort of boot (*b*) making connection by wrapping it with wire or cord with a suitable discharge pipe. Beginning at the edges of the canvas, concrete (*f*) is dumped on and rammed down firmly, crowding all the water toward the center, and allowing it to escape through the discharge pipe *d*. The lower this discharge pipe is kept, of course, the less head will be felt under the canvas. The cloth completely protects the concrete from washing, and, when set, and all filled up around the pipe, the stopper *e* is driven into the pipe, and the leak is overcome. This method seems to have some advantages over cement or concrete in sacks, and the cement is equally well protected. The close fitting of the canvas to the bottom is not a matter of surprise, or that it should be sufficiently tight to exclude water from passing beneath it; for instance, on a gravel bottom. If the head be 12 ft. for example, each square foot of canvas pressing on the bottom will have a pressure on it of $12 \times 62.3 = 748$ lbs., quite enough to insure a tight fit.

It is observable that when the pressure comes upon these canvas-covered bulkheads, the cloth fits so closely as to show the slightest inequality of the surface beneath, the total pressure, of course, if the dimensions of the bulkhead be large, being measured by many tons. On the lower lock bulkhead, the total pressure on the canvas measured over 300 tons.

The following is the cost of this bulkhead, including all cost of putting in and removing :

Lumber	$374 73
Canvas	95 00
Pipe	8 16
Diver	173 12
Nails	21 00
Labor	486 99
Total	$1 159 00
Deduct material saved	400 00
	$759 00

The bitter cold weather, of course, increased the expense. The river was frozen up when the bulkhead was removed, and the rails had to be hoisted up the lock walls and hauled to the railroad by a circuitous route. The cost of loading and unloading the rails at the railroad yard, whence they were borrowed, was considerable.

The driver was employed only five days, the removal being accomplished without his help.

DISCUSSION.

S. Bent Russell, M. Am. Soc. C. E.—Mr. Meigs' interesting paper recalls a case in my experience during the fall of 1890.

In building the new inlet tower of the low-service extension of the St. Louis Water Works, it was necessary to sink a coffer-dam on rough rock bottom. The dam was rectangular and about 26 x 62 ft. inside and 20 ft. high. The depth of water ranged from 11 to 20 ft. during construction.* The lower courses of the coffer-dam were framed together on the river bank and then floated to position.

As near as I can remember, our experience with canvas was as follows:

As it had been decided to try canvas to close the joint around the base of the dam, the contractor attached a canvas curtain to the out-

* For further information about this work see *Engineering News*, July 4th, 1891.

side of the structure. The upper edge of the curtain was secured to the wall several feet above the lower edge of the coffer-dam.

The intention was to lay the lower edge of the curtain on the rock bottom and weight it down with stones. While the dam was being floated into position, the curtain was rolled up and held by light fastenings.

When the coffer-dam had been sunk to its place, divers were sent down to spread the curtain. They reported that the canvas had become so badly damaged as to be worthless.

At the time this destruction was thought to be due to the fluttering action of the swift current in the channel of the Mississippi, and, perhaps, also, to the cutting action of sand carried by the stream.

It is possible that this result might have been avoided by proper precautions.

The coffer-dam was finally made tight by the use of bags of concrete and of clay, but the expense and delay were excessive.

In view of this experience it would be well to take extra precautions in using canvas in swift currents. As Mr. Meigs makes no mention of trouble from swift currents in the use of canvas, I have brought out this case in the hope that he will give further information bearing on the point.

Montgomery Meigs, M. Am. Soc. C. E.—The experience of Mr. Russell with a canvas curtain for making a tight joint between coffer-dam and rock was unfortunate, but the cause of the failure is not explained. Possibly the canvas was torn against a barge or some such obstacle. I do not think, if properly secured, the water would tear it.

The chain we sewed to the edge of our curtain was much better for obvious reasons than weighting with stones. On one occasion the water fell outside the dam described, and the pressure inside (the dam being at the time full of water) lifted the structure like a valve, letting the water pass underneath. The canvas and sustaining framework resumed their position perfectly as soon as the pressure was equalized, and left the tightness unimpaired.

When we put the coffer-dam in, there was practically no current and no trouble in spreading the sheet of canvas. In swift water, extra precautions would have to be taken, the canvas rolled up carefully until the time of spreading it came, battens nailed on wherever it showed a tendency to bag in the current. The divers spread many such sheets of canvas on wrecks in the Mississippi every year. Some of them 12 ft. wide and over 200 ft. long, and in a swift current.

At one wreck I assisted in raising, the bottom of the boat was ripped for 40 ft., leaving a hole 2 ft. wide in places. The diver, after the boat was raised by the usual method of surrounding the hole with a canvas and plank coffer-dam and floating the boat, went underneath her bottom, nailed 1-in. boards across the break, and spread over that a

sheet of canvas well battened. This allowed the boat to be pumped dry, and she made a trip of over 100 miles up stream, to be docked and repaired.

A swift current will, of course, always greatly increase the difficulty in placing a coffer-dam of any kind. I think that with a due appreciation of the difficulty beforehand, canvas can be safely and economically used, as Mr. Russell designed doing at the St. Louis water works crib.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

708.

(Vol. XXXI.—May, 1894.)

GENERAL NOTES ON THE GREAT KANAWHA IMPROVEMENT, TRIPPING BARS AND IMPROVED HURTERS ON CHANOINE WICKET DAMS, ETC.

By ADDISON M. SCOTT, M. Am. Soc. C. E.

READ MAY 2D, 1894.

WITH DISCUSSION.

The following was in the main prepared as a discussion on the paper of F. Guillain, Inspecteur Général des Ponts et Chaussées, on "Navigation Works Executed in France from 1876 to 1891," *Transactions*, Volume XXIX, No. 604, July, 1893, but being too late for the discussion, is now, at the suggestion of the Secretary, presented as a separate paper.

Reference is made on page 16 of the paper referred to, to the use of the Pasqueau hurter by Colonel Merrill, on the Ohio, at Davis Island Dam. As the first use in America of this valuable improvement in Chanoine dams was on the Great Kanawha, and the most extended application of its leading principle either in Europe or America has been on the same river, an account of the experience with it on the Great Kanawha seems worthy of record in this connection.

Only a brief outline of the Great Kanawha improvement will be attempted in this paper. The United States Engineer Department has recently published full detailed drawings and descriptions of lock and dam No. 7, one of the last movable dams completed, and also of lock and dam No. 2, one of the two fixed dams in the upper part of the river. The former is accompanied by a general history and description of the river, and of its commerce and improvements, with cost of construction, operation, etc. This is also published as part of the report for 1892, of Colonel William P. Craighill, the officer in charge of the work.*

The improvement of the Great Kanawha River will consist of ten locks and dams, carrying the slack-water from the mouth of the river, a distance of 90½ miles. The two upper dams of the system (Nos. 2 and 3 of profile) are fixed, and the remaining eight, numbered from 4 to 11, inclusive, are "movable," being kept up in low and down in medium and high stages of the river.

The improvement is designed mainly for a coal commerce, the barges for which are mostly 25 x 130 ft., with draft of 6½ ft., and capacity of 500 tons. The minimum low-water depth on mitre-sills is 7 ft.

Some of the important features and dimensions of each lock and dam are given in the following table :

Number of lock and dam.	Style of dam.	Maximum lift in feet.	Length of dam. Feet.			Lock dimensions. Feet.		Location, miles from mouth.	
			Navigation pass.	Weir.	Total.	Clear width.	Length between quoins.		
No. 2..	Fixed.	12	.....	.....	524	50	308	85	Finished in 1887
No. 3..	"	12	.....	.....	564	"	311	80	" 1882
No. 4..	Movable.	7	248	210	458	"	300	73¾	" 1880
No. 5..	"	7	250	265	515	"	300	67¾	" 1880
No. 6..	"	8¼	248	310	558	55	342	54¼	" 1886
No. 7..	"	8	248	316	564	"	"	44¼	" 1893
No. 8..	"	8¼	248	292	540	"	"	36	" 1893
No. 9..	"	6½	248	300	548	"	"	25½	Begun in 1893
No. 10..	"	7½	248	292	540	"	"	18¾	" 1893
No. 11..	"	10	300	372	672	"	"	1¾	" 1893

Locks and dams Nos. 4 and 5 were completed and put in operation in July, 1880, and were the first movable dams for slack-water improve-

* Report Chief of Engineers, 1892, III, 2053.

ment in America.* Five others, three movable and two fixed, as shown by the table, have been built since 1880, and three more movable dams, carrying the improvement to the mouth of the river, are now under construction.

These eight movable dams are all of the Chanoine wicket type, both on navigation passes and weirs, operated from service bridges. The pass wickets are 3 ft. 8 ins. wide—4 ft. between centers—and from 13 ft. 5½ ins. to 14 ft. 1 in. long. The vertical heights (with two exceptions, one of which is 12 ft. 6 ins., and the other 12 ft. 9 ins.) are all 13 ft. above pass sills. These wickets, with the exception of the La Mulatière dam at Lyons, France, where the vertical height above sill is 13 ft. 1½ ins., and length of wicket 14 ft. 3½ ins., are the largest ever built.

The navigation passes at dams 4 and 5 were built to be lowered by tripping bars operated by gearing wells in the lock wall and center pier on the regular Chanoine plan so extensively practiced on the narrower passes in France. The bar operated from the lock tripped 34 wickets, and was 140 ft. 10 ins. long; the one from the center pier was 119 ft. 2 ins. and tripped 28 wickets. A good deal of trouble was had with the tripping bars, from the first. Besides the difficulties unavoidably connected with them, it was found that both bars and gearing had been made too light for such wide passes. As was to be expected, the bars became clogged more or less with water-soaked sticks, cinders, gravel, etc.; when much clogged the gearing was not strong enough to pull the bar in (the wickets were, of course, tripped by the pull), and they were sometimes buckled and broken in being pushed back to place. Proper provision had not been made for keeping the bars on the guides, and they were frequently found off the track when the dam was to be raised. Any one particularly interested in the subject will find quite full reports of the manœuvres of the Great Kanawha dams each year since their completion, with accounts of the difficulties met with, in Col. Craighill's annual reports to the Chief of Engineers.

On account of the trouble with the tripping bars at dams 4 and 5, the writer, as assistant engineer on the work, in a report made in June, 1881, recommended a trial of the Pasqueau hurters on one of the bar sections, with a view, if successful, of adopting them and eliminating the tripping bars altogether.† This was approved, and the same fall

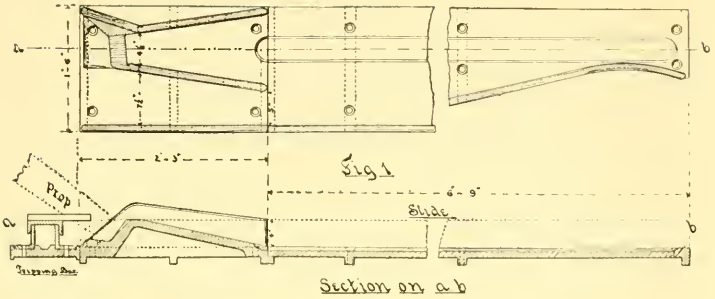
* The Davis Island dam was completed in 1885.

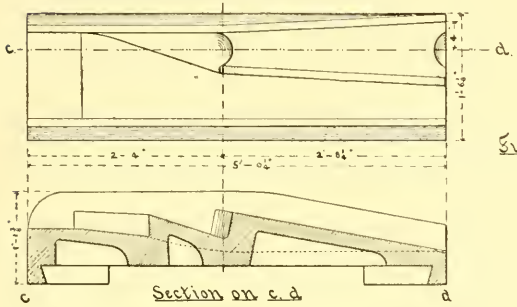
† Report Chief of Engineers, 1881, I, 914.

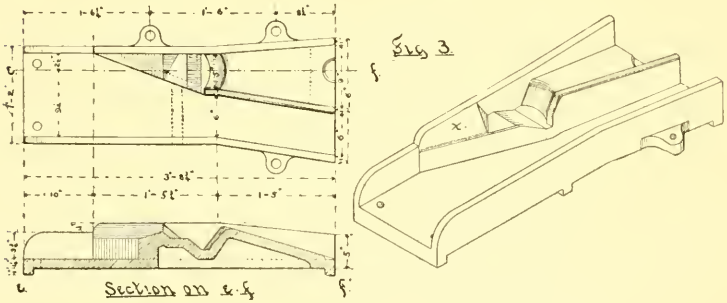
the bar on the lock side at No. 5 was taken out and some of the adapted Pasqueau hurters (Fig. 3, Plate LXXVII) put in. They were entirely successful, and since then the pier bar at No. 5, and both lock and pier bars at No. 4, have been shortened and improved hurters put in. In making the change, the Chanoine hurters were removed and the new hurters placed by divers (the regular lock hands doing the diving) without a coffer-dam. No change was made in the "slides," the same ones being used as with the Chanoine hurters.

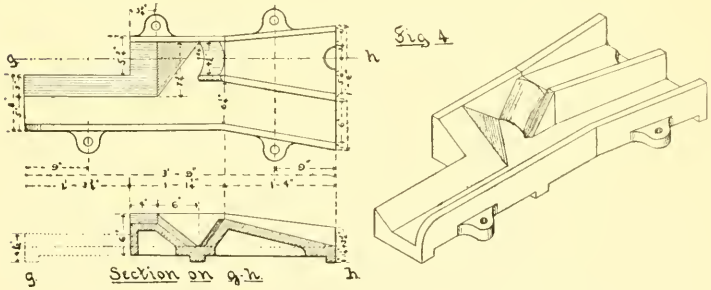
At dam No. 5 on the pier section of the pass, 16 wickets are now tripped by the bar; at No. 4, 13 wickets on the pier and 14 on the lock section are tripped by the shortened bars. These short bars give comparatively little trouble, and, being of considerable advantage in lowering, have been retained on these first dams. It may be stated here that while the decided superiority of the Pasqueau principle has been thoroughly established, experience on the Kanawha proves that it would be practicable to operate a 250-ft. pass, and somewhat wider—up to at least 300 ft., it is believed—with properly constructed tripping bars. The difference in first cost between tripping bars and the improved hurters is comparatively small, being about $2 000 in favor of the hurters (allowing for the bars and gearing to be made strong enough) on a 248-ft. pass. While the dam can be lowered faster and easier with the bars, the difference in time is not enough ordinarily to be material. The 62 wickets on one of the Great Kanawha passes, fitted entirely with the automatic hurters, are usually lowered in less than an hour. Generally about 45 minutes are taken for this part of the manœuvre; with the bars working well, they could be lowered in 10 or 15 minutes.

The great advantage of the Pasqueau principle is in time and labor saved in raising the dam. With the tripping bars, under the most favorable conditions, it is generally necessary to clear the gearing wells of deposit, and always advisable to examine the bar and hurters throughout to see that they are clear and the props down in the hurter seats in position to trip. The greater part of this work must be done by divers. At dams 4 and 5 on the first raising in the spring, when there is always considerable deposit on the foundations and in the wells, it usually took three or four days, and sometimes longer, to clear and adjust the bars and gearing to get ready to raise. After the first raising in the spring, from one to two days were generally required for this. With the improved hurters this is all avoided, and the dam in conse-


a
b
Prop
Slide
Fig 1.
Section on a b


c
d
Fig 2.
Section on c d


e
f
Fig 3.
Section on e f


g
h
Fig 4.
Section on g h

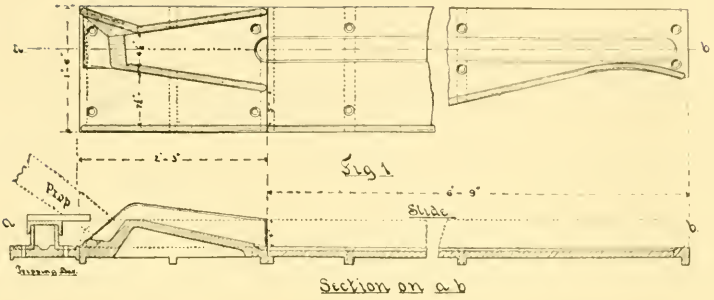


Fig 1

Section on a b

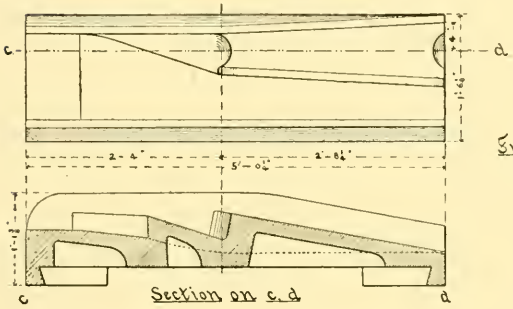
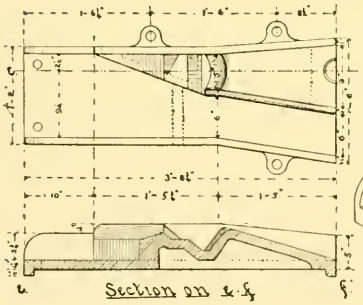


Fig 2

Section on c d



Section on e f

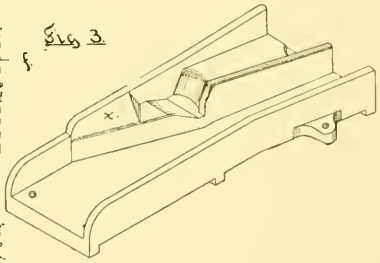
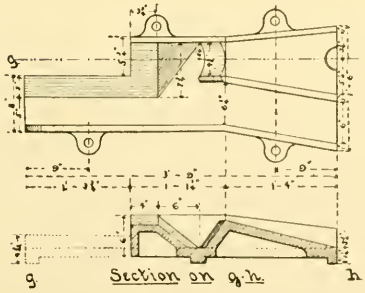


Fig 3



Section on g h

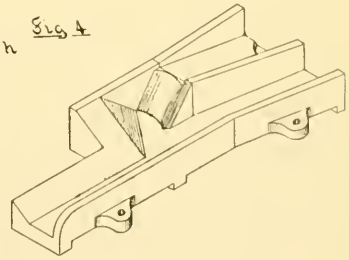


Fig 4

quence put up quicker and easier and on a higher stage of water than with the bars.

On the whole the advantages of the automatic hurters are so decided that it seems safe to say that the day of tripping bars on new Chanoine dams is past, save, perhaps, in short lengths (as at present at dams 4 and 5), and in exceptional cases, where rapid lowering is of great importance.

Without departing from the leading and important feature of Pasqueau's stepped hurter, namely, that of disengaging the prop from the hurter-seat by a short up-stream movement of the wicket, the form of the hurter has been considerably modified on the Great Kanawha. Different forms of hurters are shown on Plate LXXVII.

Fig. 1, Plate LXXVII, shows a regular Chanoine hurter in plane and section. This is the hurter used on dams 4 and 5 with the tripping bars, and is substantially the same used on the Chanoine dams in France. The tripping bar and the end of a prop are represented in the section.

The "slide" or guide below the hurter, shown on Fig. 1, is practically the same with all the hurters. The hurter and slide are often cast in one piece.

Fig. 2, Plate LXXVII, is the regular Pasqueau stepped hurter copied from an official drawing of part of the La Mulatière dam, dated October, 1879, about two years before the completion of that work.*

Fig. 3, Plate LXXVII, shows the modified Pasqueau hurter first used on the Great Kanawha on parts of the navigation passes at dams 4 and 5. The first were placed, as before mentioned, at No. 5, in 1881. The block or stop x on the top of the step was required at these dams to insure a short defined run of the prop. This was necessary on account of the nearness of the horse, when upright, to the top of the sill—the horse in this position having an inclination up stream, as usual in tripping-bar construction. The stop proved an advantage in reducing the time to lower and this form was adopted for dam No. 6, though not necessary on that work, owing to the down-stream inclination of the horse (see Fig. 5, Plate LXXVIII).

Fig. 4, Plate LXXVII, represents the "shunting" hurter adopted throughout on dams 7 and 8, both completed last year, and proposed

* The Pasqueau hurter was patented in the U. S. in 1880.

for the three remaining dams now under construction. Some of these hurters are also in use on dams 4 and 5; the first were placed for trial at No. 4, in 1888. Other forms of the shunting hurter are also in use at No. 4, but it does not seem necessary to represent them here.

The operation of this hurter is apparent from the drawing. When the wicket is pulled up stream the end of the prop is carried or shunted directly away from the seat, on the level floor of the hurter, into the descending channel, in position to let the wicket go down. This hurter works as well in every respect as the stepped form and has some, though not highly material, advantages over it. It being lower than the stepped hurter (it is necessarily no higher than the Chanoine hurter) it lets the wicket lie closer to the platform when down. It is also simpler in form and somewhat lighter than the stepped hurter.

The manner of lowering the pass wickets followed on the Great Kanawha was devised the first season the Pasqueau hurters were used on this river, and will be briefly described.

In lowering the dam as first practiced with the improved hurters, the wickets were put on the swing (*en bascule*) and then pulled up stream far enough to free the end of the prop from the hurter seat and then lowered—all being done by the winch and butt chain. The Kanawha practice is to pull the standing wicket by the top with a clutch and line till the prop is free, then to disengage the clutch and let the wicket fall. Both ways of lowering will be understood from Plate LXXVIII, representing a general cross-section of the navigation pass of dam 7. The clutch is simply placed over the top of the wicket. It has two lines as shown, the main line going to the drum of the manœuvring winch, and a smaller one attached to the upright piece of the clutch and held in hand. As soon as the wicket is wound far enough up stream to free the prop, the small line is snubbed to the service bridge and the main line released of its strain by unwinding a little on the winch. This instantly disengages the clutch and the wicket falls. This is a much better way to lower than by the chain. It can be done in about one-fourth the time, saving from two to three hours in lowering the pass (enough frequently in a quick rise to be of great importance), and avoids the risk of water-soaked sticks and drift being drawn against the horse when the wicket is swung, and preventing the latter from going down flat to place behind the sill. No injury is ever done the wickets or any of the parts by letting them fall in this way, even

Natural River Bed
Gravel and Boulders
Hard Pan
Concrete
Concrete
Hard Pan
Bed Rock
Concrete
Hard Pan
Natural River Bed
Hard Pan

in low stages with full head. The wicket starts rapidly, of course, when first released, but the water below soon checks it, and it settles down on the cushions without any noticeable shock.

The following from the published description, before referred to, of lock and dam No. 7, and of the river and general improvement, will perhaps be useful here. The description was written in 1892, before the completion of locks and dams Nos. 7 and 8.

"The experience with movable dams on this river has on the whole been very satisfactory. They are easily and rapidly manœuvred (in these respects dam No. 6 and those now under construction have considerable advantage over those first built), the expense of operation and maintenance is but little, if any, more than with fixed dams, and they prove highly satisfactory to the river interests.

"The movable dams are kept up whenever there is not water enough in the river for coal-boat navigation and down at other times. Their advantages over the ordinary fixed dams for a commerce and river like the Great Kanawha are decided, furnishing the benefits of the usual slack water without its most serious drawbacks. With fixed dams everything must pass through the locks; with them navigation is entirely suspended, too, when the river is near or above the top of the lock walls. With movable dams the locks are only used when the discharge of the river is so small as to make them necessary. At all other times they are down, practically on the river bottom, out of the way, affording unobstructed open navigation. This is of great advantage to all classes of commerce, and is particularly so with coal, transported, as it is, and empty barges returned, in 'fleets' of large barges. More barges can, of course, be taken by a towboat, and much better time made by all kinds of craft in 'open river,' when there is water enough for such navigation, than when the stage or discharge compels the use of the locks.

"The movable dams being down in high water, there is comparatively little difficulty in protecting the banks about the works from scour. In this respect they have considerable advantage, too, over the fixed dams.

"Experience with the dams has naturally suggested improvements, and No. 6, the last one completed, has considerable advantages over those first built in strength and durability of construction, facilities for rapid manœuvring, and cost of operation and maintenance. Dams 7 and 8 have been still further improved in some of their details.

"No. 6 has been in operation over five years. The average cost of operating and maintaining the lock and dam has been $2 515 per year. This covers wages, supplies, repairs, including considerable addition to the rip-rapping, and all expenses connected with the work. The entire cost during the five years of repairs on the dam proper and on

all of its apparatus, including paints, one of the principal items, has been something less than $250, or an average of $50 per year.

"This dam is put up by four or five men in from 7 to 12 hours; the usual time is about eight hours. It is lowered with the same force in about two hours. No material difficulty has ever been met with in any of the manœuvres at No. 6.

"Four men are employed regularly at each work, the same as at the fixed dams. In raising and lowering the dams, one or two extra men are often hired."

The writer has been collecting notes for a paper on the Great Kanawha works of more technical interest, especially relating to experience in construction of the locks and dams, which he hopes to have the honor of submitting to the Society at a future day.

DISCUSSION.

William P. Craighill, Pres. Am. Soc. C. E. (by letter).—The last paragraph,* page 16, of Mr. Guillain's paper on "Navigation Works in France" is calculated to give an erroneous impression and for this reason I have concluded to add a few words to Mr. Scott's paper. I have hesitated to say anything because the Pasqueau hurter or other device for manœuvring movable dams is quite limited in application, and the subject may not be of very general interest to the engineers of this country at present.

My first knowledge of the Pasqueau hurter was gained in 1878, when I saw a model of it in M. Pasqueau's office at Lyons. I was visiting that city with Colonel Merrill and Mr. W. R. Hutton, to obtain information to be applied in the development of the use of movable dams on the Ohio and Great Kanawha Rivers. I was very favorably impressed with the Pasqueau hurter, but did not think it of sufficient importance to justify a revision of the plans of dams 4 and 5 on the Great Kanawha, which were then, and had been for some time, in process of construction. These dams were provided with the usual tripping bar of the Chanoine system, which had answered well for navigation passes of moderate width.

The tripping bar on the Kanawha did not turn out as well as was expected, owing mainly to want of due proportion of parts and consequent want of stiffness and strength. It was therefore decided, some time after my return from Europe, to make a trial of a few of the Pasqueau hurters on the Great Kanawha. This was done, and, as Mr.

* *Transactions* Am. Soc. C. E., Vol. XXIX, p. 1.

Scott remarks, "the first use of this valuable improvement was on the Great Kanawha, and the most extended application of its leading principle, either in Europe or America, has been on the same river."

M. Pasqueau had his invention patented in this country, and, through agents, made a claim for a large royalty. His claim was considered excessive, especially as his invention had been used without charge in France and its experimental introduction in this country had largely advertised it. A reasonable offer was made for the use of the invention in this country, and declined. Whereupon suit was brought in the Court of Claims by M. Pasqueau's agents.

These difficulties as to the continued use of the Pasqueau hurter and further experience on the Great Kanawha led to the invention of what should properly be called the Scott hurter, which is now exclusively introduced into the later dams of the Kanawha. It is fully described in Mr. Scott's paper, but he is too modest to claim all the credit which is his just due. It gives me great pleasure to award it to him.

On another occasion I used the language below concerning Mr. Scott, which is entirely true:

"I found him on the Kanawha when I took charge of it. He has been either the Principal Assistant Engineer on the Kanawha, or the Resident Engineer, almost, if not altogether, continuously since that time, I think, and has had more to do with the superintendence of the construction and operation of the movable dams on the Kanawha than any other person, and my opinion is that his experience, in comparing the Pasqueau hurter with the tripping bar, is larger than that of any other man in the world."

Under the supervision of Mr. Scott several modifications of the details of the manœuvres of the dams have been made, and all in the line of improvement.

J. P. Frizell, M. Am. Soc. C. E. (by letter).—The movable dam is one of the most remarkable developments of modern hydraulic engineering. There are probably few engineers of any experience in river work who have not, at times, been impressed with the great advantage that would result from the ability to set up a dam in low stages of the river, and leave a free passage for the water in times of flood. The introduction of this form of construction in the United States has been attended with great opposition, an opposition founded for the most part upon misapprehension. The idea of maintaining a 10, 12 or 15 ft. head of water by a series of light wooden rickets resting against props appears ridiculous to people who are accustomed to regard a solid mass of masonry, as wide as it is high, as none too strong for that purpose. They fail to consider that the forces which act distinctively upon a dam are created by the dam itself. A mass of water is formidable only when set in violent motion. A movable dam is largely free from the dangers to which fixed dams are exposed. It exists only when

the flow of water is slight. It controls the stream only in its times of tameness and docility. It lies safe and harmless on the bed of the stream when the latter is wild and violent.

These remarks apply more especially when the movable dam extends entirely across the stream. When it simply controls a pass in a fixed dam it is liable to occasion a scour more dangerous than if the entire dam were fixed.

The chief advantage to navigation contemplated by the use of the movable dam is that it subjects vessels to the delay of passing the lock only during the lowest stages of the stream, leaving a free channel when the flow of the stream is sufficient for the requirements of navigation without the aid of slack water. Also that on streams liable to great freshets it admits of a cheaper construction in not requiring lock walls and the abutments of the dam to go above high water.

Four general types of the movable dam have been used to a greater or less extent in works of navigation. The first to be mentioned is the bear trap gate, of which all engineers have a general idea. This consists of two leaves extending entirely across the channel, turning on horizontal parallel hinges, attached to the floor of the channel. The taller of the two leaves is on the up-stream side, and, when the channel is open, lies folded over the shorter. Water from a higher source, admitted to the chamber under the gates, causes them to rise, and, being withdrawn, causes them to fall. The top of the shorter leaf remains in contact with the side of the longer one, and they form in connection with the side walls and bottom of the channel a closed space sufficiently water tight for the purpose. The bear trap gate is an American invention, and was first applied by Josiah White, of Pennsylvania in 1818, as a lock gate on the Lehigh Canal. It has since found many applications as a movable dam, properly so called, on the rivers of Pennsylvania. In May, 1887,* a board of engineer officers reported that dams on this principle were perfectly practicable for openings of 60 ft. and a lift of 12½ ft. This construction was applied to dam No. 1 on the Monongahela River,† about 1880, to an opening of 120 ft. The difficulty here encountered was that the dam did not rise uniformly, one end rising considerably in advance of the other.

This device exceeds every other form of movable dam in the ease with which it is manipulated. It has no disadvantages that could not be overcome by perseverance and intelligence. It is to be regretted that it has not been more extensively applied. The construction of such a dam was commenced several years ago at Beattyville, on the Kentucky River, but was abandoned for reasons wholly unconnected with its merits.

The second system is the Thenard shutters, first applied by

* Report of Chief of Engineers, U. S. A., 1887, Vol. 3, p. 1882.

† Report of Chief of Engineers, U. S. A., 1885, Vol. 3, p. 1862.

Thenard on the River Isle, in France, in 1828. This consists of two parallel sets of shutters extending along the top of a low masonry dam to which they are hinged. When the channel is open, they lie extended on the masonry, each set falling from the other. The up-stream set is confined by latches releasable by a tripping bar. When the dam is to be raised, the attendant releases the up-stream set and the current raises them into an erect position. This stops the whole flow of the stream and holds it in the pool above the dam, leaving the masonry accessible, so that the down-stream set of shutters can be erected and confined by props. When the water rises to the top of the up-stream set, and the pressure comes on the down-stream set, the former fall and are automatically latched. When the channel is to be opened, the down-stream shutters are dropped by a tripping bar.

This system appears to have worked admirably in the cases to which it has been applied. In the application described it did not serve as a dam, but as what we term flash boards. It is manifest that, applied to a navigable channel, the lower shutters would not become accessible when the upper ones are raised; but with suitable modifications, the principle is perfectly applicable to that case.

The third system, and one which has found extended application on European rivers, is the Poiree or needle dam. In this the framework of the dam consists of a series of trestles, which, at the same time, sustain a bridge for the convenience of workmen. The water-tight diaphragm consists of planks or joists called "needles," which are brought on to the bridge from a storehouse on shore, and are inserted from above. When the channel is open, the trestles lie flat on the floor, their feet resting in bearings whose axes extend up and down stream. They are raised by chains worked from the wall or abutment. The objections to this system are mainly the difficulty of maintaining a sufficient degree of tightness, and the large amount of material to be handled at each manœuvre of the dam.

The fourth system is the Chanoine, which forms the subject of Mr. Scott's paper. In this, each trestle carries the corresponding part of the water-tight diaphragm of the dam, called a wicket. The feet of the trestle rest in bearings whose axes are transverse to the current and in line with the dam. The top of each trestle has two journals to which the wicket is articulated, and a third to which is jointed the prop for supporting the trestle. When down, the trestle and its props lie extended in the same line on the floor, and the wicket lies over them. A strong pull on the up-stream end of the wicket raises the trestle together with the wicket, the prop following till it reaches a notch or step in the floor, when the whole articulated system rests firmly, the wicket hanging suspended near its middle part. Depressing the up-stream end of the wicket, it rests on a suitably formed sill, the wicket standing in a nearly vertical position and forming a part of

the dam. A bridge formed by trestles similar to those of the Poiree dam, though lighter, is usually set up for convenience in erecting the Chanoine dam, though the work may be done from a boat held by cables. The wickets, of course, cannot touch one another or approach within less than from 1 to 2 ins. These openings are most conveniently closed by square scantlings which rest in them with the diagonal vertical. These scantlings are the only extraneous parts of the dam. The trestle is thrown down by releasing the foot of the prop.

This brings us to the principal point of Mr. Scott's paper, which deals specially with the methods of accomplishing this result. Two methods are in use for that purpose. The first consists of a tripping bar, which is a stout iron rod reaching the whole length of the movable dam, and susceptible of a longitudinal movement by means of gearing at the lock or abutment. Projections, or studs, on this rod successively engage the foot of each prop and draw it off its seat. These studs are spaced a little wider than the wickets, so that the tripping bar engages with but one prop at a time, though, of course, the studs can be so spaced as to trip any number of wickets at once. The common arrangement is to throw down singly till the pressure is sufficiently reduced, then in two's, then in three's, etc. This, I understand, is the method not found satisfactory on the Kanawha. The second method consists in a peculiar construction of the plate on which the foot of the prop slides, in raising and lowering the trestle, which plate has retained its French name of hurter. It is so formed that the foot of the prop pursues one path while the trestle is rising, and another while it is descending. In the first path there is a notch or step into which the foot of the prop falls and rests firmly when the trestle has reached its proper position. When the trestle is drawn further forward, the foot of the prop follows, and reaches a position when its path terminates. It slides sidewise into the second path which offers no obstruction to its backward movement, and allows the trestle and wicket to fall freely. When the trestle is down, the foot of the prop finds itself again in the path which it follows while rising.

This style of movable dam is the one that has found greatest acceptance in this country. Whether it is the best that can be devised is a question of minor importance. That it effectually and practically fulfills the requirements of a movable dam no longer admits of doubt. It is not inordinately expensive, and its cost may be expected to diminish with extended use and experience.

It appears to me that a great error was committed by the United States Engineer Department in not adopting movable dams for the Cumberland River. The stream is navigable some six months in the year as far as Smith's Shoals, some 350 miles above Nashville, in Tennessee. For a long distance above Nashville its slope is 8 ins. to the mile, and still less below that place. It flows in a cañon worn out

of the rock 100 ft. or more in depth, half filled with alluvial deposit. It is liable to extreme floods of over 50 ft., and rises of 35 ft. are not rare. To raise lock-walls and abutments above high water is out of the question, not only on account of the expense, but on account of the contraction of the flood water-way. The locks must often be wholly submerged and out of use. With permanent dams there cannot fail to be times when boats can neither pass the lock nor dam. Like all the smaller western rivers, this stream is frequented by stern-wheel steamers. A system of slack-water navigation is contemplated for this stream, involving some 30 locks and dams of 10 or 12-ft. lift. A rise of 35 or 40 ft. would, no doubt, make nearly smooth water over a 10 or 12-ft. dam. But with a rise of 20 or 30 ft., which would put the lock out of use, there would be a fall at the dam such that a stern-wheel steamer going up would on reaching the upper level find its wheel out of the water, helplessly beating the air. No river in America presents more distinctly the conditions calling for movable dams.

Three boards of engineer officers have considered this question. The first recommended fixed dams; the second advised movable dams; the third reversed the latter decision and redecided for fixed dams. No dams, however, will be required till several of the locks are completed, and there will be abundant time to revise the latter decision.

Almost every dam built for industrial uses calls for some application of the principles of movable dams. Most such dams are liable to frequent floods, giving a depth of 6 ft. or more on the dam. This height affects the lands bordering the streams as far up as the influence of the dam extends. The damages incident to such a rise of the water have to be paid in obtaining the right to build the dam. These damages are but very slightly aggravated by holding the water permanently at the ordinary flood level, to the great advantage of the water-power in respect to fall and pondage. The permanent part of the dam cannot be built any higher, as that would increase the flood height. The conditions call for some addition to the height of the dam, which can be erected in low water and dropped on the approach of a flood. The arrangement of flash boards now in use is a very crude expedient, very imperfectly meeting the requirements of the case. It consists of loose boards held by the pressure of the water against pins inserted in holes drilled or bored in the top of the dam. This arrangement is limited in height, never exceeding 4 ft. The obstruction is not removed in time of flood, except in so far as the pins are broken or bent and the boards carried away at such times. It has often appeared to me that, in a dam constructed with reference to this requirement, this exceedingly primitive and imperfect device might very advantageously be replaced by something like the Thenard shutters above referred to. More especially are such arrangements applicable on southern rivers, where ice, that implacable enemy of all refinements in river construction, does not exist.

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709.

(Vol. XXXI.—May, 1894.)

STORAGE AND PONDAGE OF WATER.—DISCUSSION ON PAPER No. 688.*

By WILLIAM E. WORTHEN, A. FTELEY and CHARLES B. BRUSH.

WILLIAM E. WORTHEN, Past President Am. Soc. C. E.—The facts collected by the author of the paper are right, but to undertake to form a general rule of yearly averages of water flows I object to, unless nothing better can be got. To an expert in water suits the best evidence of the capacity of a stream is from the daily flows for as many years as possible, plotted in an extended profile, and, on this, continuous horizontal lines of different flows per day, from which can be determined how much can be obtained daily through the whole profile, what pondage will be necessary, reference being had to what can be readily secured, discarding excessive flows which raise the average of flows so much, and cannot be ponded. On the lower Bronx there are three months in the year in which there is little, if any, power to be obtained without pondage, and the necessary land for pondage can not be got.

There is another matter not referred to in the paper, but pertinent; that is, the expert testimony in the cases of diversion of water, which is almost as conflicting as medical testimony. My aim is to be consistent

*"Storage and Pondage of Water," by Joseph P. Frizell, M. Am. Soc. C. E. Vol. XXXI, p. 29.

on whichever side I am retained, and inform my client of my view of his case, and especially of the points on which I cannot support him; but expert testimony should be for the court, and not for clients.

Mr. Fteley.—I recollect an instance which illustrates the remark just made, that expert testimony should be for the court and not for the clients.

The case on hand was of some importance, and experts were engaged on both sides; but, although their testimony could not be objected to as regards fairness, the facts before the court were so presented that important points were omitted, thus favoring an appearance of contradiction between the experts on each side. From conversations subsequently held between the experts, with all the facts at hand, I do not doubt that, if consulted by the court, with all the evidence before them, they would have substantially agreed.

Charles B. Brush, M. Am. Soc. C. E.—It seems to me that Mr. Fteley has pointed out the cause of a great deal of the difference that appears in expert testimony. When experts are asked to give their opinion on a subject, and are asked in a general way, so that they are free to give that opinion, that is one thing; but as a rule that is not the way they are asked their opinion in causes of this kind. Almost invariably there is an assumption made, and, based on that, they are asked what results follow, and they are confined by the very nature of the question that is asked them.

Mr. Worthen.—What effect has the cross-examination?

Mr. Brush.—The cross-examination does not always bring out the facts on the other side of the case. It is true, that if an expert should really be given the opportunity of saying frankly what he believes on any subject, a great deal of this difference would disappear; but the expert is confined to certain lines of examination, so that the cross-examination does not always obviate this difficulty. He gives part, but, as in the case that Mr. Fteley brought up, the value given by the experts on the basis of an assumed condition was a certain amount. On the other side, on another assumption, a different result was reached. Afterwards, when the experts came together, there was no difficulty in coming to an agreement, taking all the facts as they found them.

It is most unfortunate that experts are confined to certain lines to which the lawyers on either side feel that the interests of their case requires that they should be confined.

Mr. Fteley.—I wish to ask Mr. Worthen a question.

If you are retained by one party, and, notwithstanding your desire to give true testimony, even if it is to injure your client, it may happen that you know of weak points, unknown to the other party, and which, if brought up, would injure your case, would you then consider, under your oath to "tell the truth, all the truth and nothing but the truth," that you are in duty bound to disclose such weak points?

Mr. Worthen.—I say, no. I do not consider that it is my duty to offer evidence of the weakness of my case at all; but I state distinctly to the lawyers on the side on which I am retained, what the position is.

In a case at Trenton, for the removal of the dam across the Delaware, the answers which I gave in court to the questions on direct examination left a wrong impression, but it was relieved by the question of the judge. And in most cases, I have found that the cross-examination or interrogatories of judge or of commissioners bring out the truth; at any rate, I depend on that. The phrase "to tell the truth, the whole truth and nothing but the truth" is construed by the court and counsel to mean truthful answers to the questions asked. It is, probably, within the knowledge of every one who has been in any court, that where a witness offers anything explanatory, for the attorneys to say that he was not asked that, and for the judge to say to him he had better confine himself to the question.

I try to make answers full and explanatory of the rules of flowing water, and decline to appear as witness where I must be considered as a retained counsel, to give one-sided evidence under oath.

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710.

(Vol. XXXI.—June, 1894.)

ADDRESS AT THE ANNUAL CONVENTION AT THE CATARACT HOUSE, NIAGARA FALLS, N. Y., JUNE 20TH, 1894.

By WM. P. CRAIGHILL, President Am. Soc. C. E.

Gentlemen of the Am. Soc. C. E.—Our Constitution prescribes one duty for the President of the Society in a very explicit and mandatory manner. It says: "He shall deliver an address at the Annual Convention." Fortunately, the length of the address is not defined. I feel at liberty, therefore, to make mine very short. It will thus have at least one merit.

Here I may remark, for the benefit of my successors, that I doubt the expediency of requiring the President of the Society to make an address on this occasion, and I would suggest a change in the phraseology of the Constitution, so that it will read "he may deliver an address at the Annual Convention," and not "shall."

After casting about for a subject, I have decided to give you some thoughts about our national defences, and to refer in the beginning to some erroneous ideas which have prevailed and do prevail among many of our people. I will first indulge in a little ancient history.

When the great civil war came on, the sea-coast defences of this country, so far as the system had been carried out, were admitted to be as good as any in the world. They were built to resist naval attacks, and with that object their general plan and detailed arrangements were excellent. The character of the materials and workmanship was not surpassed by any.

In a certain particular, one of the greatest importance, the arrangement was better than that possessed by any other people. I refer to what is called the Totten embrasure. The embrasure, as you are aware, is the opening in a defensive wall through which a large gun is fired. Without going into the details of the Totten embrasure, I will call your attention to one great improvement it introduced. While in a casemated battery built for the defence of a great European port, just before the introduction of the Totten embrasure in this country, the opening in the wall was 54 sq. ft., and allowing the gun a horizontal traverse of only 40°, the Totten embrasure gave an opening of only 9 sq. ft., while the gun had a horizontal traverse of 60°, and as much elevation and depression as were necessary. The wall around the throat of this embrasure was strengthened by the insertion in the masonry of 1 ft. in thickness of wrought iron, and there was used also lead concrete, which was found to be a most excellent material for resisting the impact of shot, and could be readily introduced into irregular spaces in the masonry.

Shutters of wrought iron were also added, so arranged as to be open only when the gun was ready to be fired, and of such a thickness as to exclude missiles as large as grape shot. The shutters were provided with springs on the backs so as to close spontaneously under the effect of the impact of the explosion caused by the firing of the gun. The smoke from the discharge was also excluded by them. This invention of General Totten, who was then Chief of the Corps of Engineers of the Army, is of historic interest as the first instance of the use of iron plating on land batteries.

Some of these old masonry forts containing these admirably arranged embrasures are still of value, though they are by no means to be entirely depended upon for the defence of our harbors. But they are still useful, as I will try to explain.

A very important part of our present system of defence consists of torpedoes, and these should be under fire to be of any value; other-

wise an enemy could readily remove or disable them. The vessels that would be chiefly used to remove or disable torpedoes would be the lighter craft which would be quite vulnerable under the fire of such guns as the Totten embrasure will still accommodate well.

An attacking fleet will not consist wholly of great ironclads. These will be met by our heaviest guns and mortars mounted behind impenetrable covering masses, and made as safe as possible by the use of hydraulic lifts, disappearing carriages, etc., but it would be a waste of energy and money to fight the smaller craft with these largest guns.

An interesting lesson for our day may be learned from the attack of the British on Baltimore in the war of 1812. On that occasion troops were landed from the fleet at North Point, about 12 miles below, and moved upon the city. Simultaneously the fleet attacked Fort McHenry, which was at that time the main defensive work for the harbor and situated on the outskirts of the city. But this was not all. The British Admiral, under the cover of night and the smoke of his bombardment, sent a number of small boats past the fort, filled with quite a force of men, who were expected to land in its rear and attack it from that quarter. Fortunately, the Americans had several small guns in a retired battery, and from this position the boats, which had safely passed the main line of defence, were fired upon, some of them sunk, and their occupants so demoralized that they retired in confusion. This was the crisis. When this boat attack failed, the fleet withdrew, the troops fell back and re-embarked, and the attack was abandoned.

So, in our day, prudence and economy require the use of many comparatively small guns suitably located, and they will be found most efficient and valuable for the purposes indicated.

The fall of Fort Sumter led to grave misapprehensions which still exist in many minds. The walls of this fort were of concrete, with an interior and exterior facing of brick. It was built to fight guns afloat and not guns ashore. It had been a principle of the art of fortification from the time of Vauban and Cormontaigne that masonry should never be exposed to the fire of land batteries. It was never expected that Fort Sumter at Charleston or Fort Pulaski on the Savannah River would be assailed from the land and from the rear, as they both were.

And it is a remarkable thing that Fort Sumter never surrendered

because of the fire of guns upon it either from the land or from ships. In the first attack upon it by the Confederates the defensive properties of the fort were not impaired and nobody was hurt. Major Anderson had no provisions and would have been obliged to surrender in a day or two if he had not been fired upon at all.

Fort Sumter was next assailed by the United States fleet under Dupont, which was iron-clad and the most powerful which had ever made an attack up to that time. After a gallant fight he withdrew, finding, as the report says, "no impression made upon the fort." The Admiral says, "in the brief though fierce engagement five out of the eight iron-clads were disabled, and a half-hour more of fighting would, in my judgment, have placed them all *hors de combat.*" It should be observed, too, that while the armament of the fleet was of the best of that day, the armament of the fort was not at all so.

Admiral Dahlgren succeeded Dupont in the command of the fleet before Charleston. He lay off the harbor for months, but never ventured on another attack; for Sumter had done what it was built to do—it had fully repelled the attack of a powerful fleet.

Later in the war Fort Sumter was much knocked to pieces by the batteries on the land and in its rear, but it was still capable of defence and was never actually captured. When Charleston surrendered after Sherman's march, Fort Sumter was surrendered with it, but it was not captured.

I was at Fort Sumter not a very long time ago. It is still valuable as a strong position against a naval attack, and will prove my assertion true if ever we are assailed by a foreign foe.

Many people think that we should strive for invulnerability in our forts. They are mistaken. The question is one of relative vulnerability and endurance, and in these particulars the advantage will be always on the side of the defence. No matter how much armor is put on ships, we can always put more on the land batteries. We can mount the same guns on shore much more cheaply than on ships, and on shore we have the great advantages of stability, accuracy of fire, and other things which are unattainable afloat.

The successes of Farragut at Mobile and New Orleans have also been much misunderstood. It is a principle of the defence of any place by forts against ships that the enemy should not be allowed to find any spot along the nearer approaches to the object to be defended,

or within extreme cannon range of it, where he will not be himself exposed to a powerful fire of many guns of the heaviest calibres, and there should also be an abundance of torpedoes over all these areas, fully protected by suitable guns bearing upon them. The defences of Mobile and New Orleans had never been completed. When Farragut had boldly passed the outer line at each place, he was no longer under a fire of any importance. The victory was won.

The contest for supremacy still continues between guns and armor. The day may come when to protect ships sufficiently to enable them to resist the shots of the most powerful guns, will make them so heavy as to be too unwieldy. To undertake to plate the decks so heavily, as well as the sides, would certainly have that effect. And to leave the decks unplated, as is practically the case at present, makes the ships very vulnerable to the missiles of the large rifled mortars, the accuracy of whose fire has greatly increased.

The time may come when, as the knights of old threw off their cumbersome armor upon the invention of gunpowder, ships will be left unarmored, and will trust in their speed and heavy guns, of which they could then carry a greater number. We may all come to the practical adoption of the saying attributed to the great admiral, Farragut, "broadsides rather than ironsides." In truth, the first-class armored warship of the present day has become a most unwieldy monster, full of the most intricate machinery. We all remember the frightful disaster to the British ship *Victoria*, which was sunk by her companion, the *Camperdown*, in the Mediterranean Sea. In 1893 six other ships of war were lost, and but one of them in actual war.

A very high authority, Mr. Cramp, the eminent ship-builder, has lately expressed the following opinion : "The most important fact of all is overlooked or not understood by the general public. All armored construction proceeds upon the admitted supremacy of the gun. No one conversant with the situation expects armor to be invulnerable.

"Naval architects do not pretend to make an impenetrable ship.

"They only try to do their best within fixed limits of the carrying capacity of the ship. The gun always has the advantage. The gun can never have so much the advantage when afloat as it has at the testing-ground" or ashore.

No armor has yet been produced that can properly be called invulnerable ; and it is reasonably safe to say that no such armor ever will

be produced, notwithstanding the strengthening due to the process of Harvey.

Having referred briefly to some things in the past, let me now dwell a little while on the present.

Let us ask ourselves what is a proper system of defence for our country, situated as we are, with a very small standing army, a very large territory not densely populated, a very long coast line, and our eastern and western sections separated from each other by wide, barren tracts of land and high ranges of mountains.

Some would say that we need neither forts nor ships, that no enemy would undertake to land an army on our shores, as our militia would rise up and drive the invaders into the ocean or capture them all. The people holding such opinions are generally those who dwell in the interior and are little likely to be attacked by anybody, or to have their property seized or destroyed. It is true the militia of the country are as brave as any men, and when converted into veterans by the discipline and experience of actual service make as good soldiers as any in the world, as our great civil war proved, and as had been demonstrated in the Mexican and previous wars. But the appearance of a foreign army in our midst must be preceded by a landing on our coast somewhere, unless it come from Mexico or Canada. We can very well trust to our army of militia to take care of an invasion by land. In case, however, of an attempt at a descent upon our coast, a proper resistance could not be made by the militia alone, however brave and well-disciplined they might be. They could make no prolonged resistance to the big guns of an invading fleet. The more men lined the shores, armed with the best rifles and with field artillery, and even if provided with Herr Dowe's bullet-proof coats, the greater would be the slaughter and disaster.

It may be safely concluded that for security against invasion we must have something more than the army to depend upon. We need something to keep an enemy's ships carrying the largest long-range guns, at such a distance from our vulnerable points as to have them secure.

The object of an enemy is always to do the most damage he can to his adversary with the least harm and danger to himself. The most valuable and easily accessible objects of an enemy's cupidity are our great cities and naval establishments on or near our coasts. And as

such establishments are generally located on the finest harbors, they become thereby most easily accessible by the largest ships of an enemy. How shall they be made secure ?

We might resort to the most obvious method, to fill up the channels of approach by such obstructions as would prevent our enemy from getting at us. But this would excite the derision of the world, would not be worthy of a brave people, and would also prevent our own ships from having access to the ocean, and thus destroy our own commerce, one of the first objects the enemy himself would desire.

Some advocate reliance upon torpedoes alone, but they forget that it is as easy to take up torpedoes as to put them down unless they are under fire. Torpedoes do very well in their place and are a necessary adjunct to the defensive arrangements of the present day, but they cannot be depended on unless supplemented by something else. This something else is guns of respectable size.

The next question that arises is, shall these guns be mounted in forts or in ships ? Some think forts unnecessary and ships all that is needed.

To depend on the navy alone would require that at every point to be defended we should have at least as strong a fleet as an enemy could be expected to bring against us, as we could never tell at what point he would appear. If our defence at one point were successful, the enemy could promptly go somewhere else, and we must still remain in our first position lest he return. If our defence at any point failed, we would lose not only our ships which were stationed there, but also the place we had attempted to defend. With such a coast as we have we could never expect to provide ships enough to defend it thoroughly. The first cost of mounting guns ashore is far less than putting them on ships, and the life of ships is short; whereas forts of masonry and sand may last for any desired time.

It would take a navy greater than that of all the other nations of the world combined to defend the immense coast line of the United States. We can never expect to have a navy one-twentieth as large as that. And it would not seem fair or right, even if we had such a navy, to tie it down to the purposes of coast defence, when its power of mobility enables it to take the more imposing duty of going after the enemy, fighting him where he is to be found and compelling him to keep near his own possessions for their defence.

As far back as 1816 the Board of Engineers laid down the following principles for the defences of our coast:

First.—They must close all important harbors against an enemy and secure them to our military and commercial marine.

Second.—They must deprive an enemy of all strong positions where, protected by naval superiority, he might fix permanent quarters in our territory, maintain himself during the war, and keep the whole frontier in perpetual alarm.

Third.—They must cover the great cities from attack.

Fourth.—They must prevent, as far as practicable, the great avenues of interior navigation from being blockaded at their entrance into the ocean.

Fifth.—They must cover the coastwise and interior navigation by closing the harbors and the several inlets from the sea which intersect the lines of communication, and thereby further aid the navy in protecting the navigation of the country.

Sixth.—They must protect the great naval establishments.

These principles are fundamental and have stood the test of argument and experience. While principles do not change, the means of carrying them out do. A half century ago use was made mainly of masonry walls in our country as in others. And the idea had very obvious advantages; but these walls do not satisfy present conditions as a main dependence, although, as stated above, there are many positions in which they are still useful and valuable.

The works arranged for the defence of important points should be so located that the enemy cannot come within range of the objects to be protected without subjecting himself to a heavy fire. The best points for the defensive works are at narrow parts of the channel of approach, or where the channel comes near the shore. The armaments should be at least equal in weight of metal to those of any possible attacking fleet, and the torpedo defence should be so complete that no ships could attempt to cross the lines without danger of destruction.

The very great range of the modern rifled gun is a decided advantage for the navy, as it enables a hostile fleet to throw shot and shells into a large object, like a city, from a very long distance. This makes it necessary for the defence to occupy lines much more extensive and much farther than formerly from the object to be defended. On the other hand, while this very great range may not be of much value in

firing at an object so small as a single ship so far distant as to be invisible or nearly so, the great power and accuracy of the rifled gun make it extremely dangerous for a ship to come within range of the shore batteries, even if she be covered with the heaviest armor.

The true, and grand, and inspiring function of the navy is to protect abroad and on the high seas our rights and our commerce, to extend the weight of our arms to the enemy's own shores, and to keep his fleets so busy in acting on the defensive as to disable them from blockading or attacking our ports.

General Abbot, the President of the existing Board of Engineers, has, in a recent magazine article, succinctly stated his conclusions on the subject, as follows:

"In fine, our true policy for coast defence is to fortify the chief ports along our coasts, so as not only to protect the adjacent cities and arsenals against insult, but also to provide safe refuges for our coastwise marine, and safe naval bases with coaling stations, depots of supply, and places of refuge for our ships of war when threatened by superior forces on the ocean, whence they may issue to act offensively as circumstances permit. This would render any attempt to blockade a wide expanse of coast impracticable, and would leave our ships of war free to operate on the lines of supply of the enemy if he should attempt to concentrate and attack any particular port."

The use of large rifled mortars enters more extensively into the American system of fortification than of any other nation as yet. By this means iron-clad ships are assailed in their most vulnerable part, their unarmored decks. With a mortar having a calibre of 12 ins., enough weight of projectile is obtained to insure penetration of the decks, and the shells have size enough for large charges of high explosives. The mortars may be mounted at comparatively small expense, so that their number can be greatly increased and to an extent more than sufficient to over-weigh their smaller accuracy of fire. In this last particular, however, the modern mortar is greatly in advance of its predecessors. These mortar batteries can be readily located in low and retired positions, so as to be out of sight of an enemy and thus much less liable to injury from his fire.

The coast defences of Charleston successfully withstood the naval attack of Sir Peter Parker in 1776. They again withstood the naval attacks of 1861–65.

The naval attack of Admiral Cockburn upon Baltimore failed in

the war of 1812. Washington would not have been captured and burned in the same war if the approaches to the city by the Potomac and Patuxent rivers had been properly defended against ships.

In the civil war Fort Fisher, on the Cape Fear River, yielded only to a combined attack of the land and naval forces.

Our defensive works have been arranged in classes according to the relative importance of the places to be defended. For years nothing was done by the United States on fortifications or for the Navy. There has been a considerable revival in these matters in the past few years, but we are still much behind European nations. In this period of inactivity on our part other nations have spent large amounts of money in most expensive experiments with guns and armor. We have profited by their experience. The eyes of our officers, both of the Army and Navy, have followed these experiments with eager attention and we have thus gained much knowledge without much expense to ourselves. The reasonable limit of size of guns and the dimensions of armor plates seems to have been reached; we have now excellent factories of our own for turning out both guns and armor, and we can now go forward in making them. The gun factory at Watervliet is not surpassed in its capacity and arrangements, which take in all the latest improvements. The same may be said of the navy factory at Washington.

The magnificent works at Bethlehem are most complete in their appointments, and we have other admirable private factories, even if one of them has turned out a lot of bad plates. The Pacific coast should have a factory of its own for producing the heaviest guns and carriages.

There is another most important element in our defensive system which deserves extensive development, and fortunately public attention seems to be turning very much in that direction. I refer to the Ship Canal.

Our Atlantic and Gulf coast is so very long that the interior line of water-way parallel to it, which could be made so easily, is of very great importance. Its advantages are great from the commercial point of view in time of peace, and in time of war with a great maritime power its usefulness to our naval and merchant marine would be incalculable.

The interior water route parallel to the Atlantic and Gulf coasts

would enable the navy to pass rapidly and freely from point to point even if the coast were blockaded, and thus, as it were, multiply itself.

Take, for instance, the advantages of a ship canal connecting the great bays of the Chesapeake and the Delaware. It would enable a single fleet to be prepared to promptly meet an enemy threatening either of these bays, and, while waiting for his plans to develop, it could remain in a secure harbor in the canal, from which it could readily emerge in either direction when necessary.

The recent experiment made by the torpedo boat *Cushing* in passing by the inland route from Washington to New York is very interesting. South of Washington, Nature herself makes the way for its easy continuance at small cost.

A very interesting description has lately been sent to me by Mr. Marshall Parks, of Norfolk, Va., of a voyage of a little steamboat called the *Germ* more than 50 years ago from Norfolk up Chesapeake Bay, through the canals to the Delaware and Hudson rivers, up the Hudson, into the Erie Canal as far as Syracuse, up to Oswego and out upon Lake Ontario. This little boat, the *Germ*, was 50 ft. long, 8 ft. beam, with a draft of 3 ft.

The voyage of the *Cushing* has shown that, even with our present canals and interior water-ways, torpedo boats could be concentrated at any point of attack without going out to sea, and, of course, be free from molestation from the ships of the enemy. But the widening and deepening of these inside passage-ways to an extent sufficient to carry our new cruisers and iron-clads to any city that might be threatened with attack is a matter still more important.

For the Chesapeake and Delaware Canal very elaborate surveys were made in years past under my supervision, and full information presented to Congress, but nothing has yet been done. It is a question which, in a commercial sense, interests western shippers by offering a shorter, cheaper and less dangerous route to Europe, and also those engaged in coastwise commerce; while to the Army and Navy, by giving unimpeded movements to fleets, to troops and munitions of war, such interior ways would present advantages that have long been recognized by both naval and military men.

The people of Philadelphia are also fully alive to the importance of the inland canal between that city and New York. In time of war it would enable the Government to move its fleets from the mouth of

the Delaware to New York in perfect safety. Then, if the Chesapeake and Delaware Canal were also made into a ship route, Norfolk, Washington and New York would be in close communication with one another. If the water front should be beleaguered by an enemy, this inside line would become almost invaluable. As for merchant vessels, there would be no storms for them to encounter, no possibilities of shipwreck. They would be in land-locked waters all the time.

England works assiduously at her water line from the mouth of the St. Lawrence to the interior of her immense North American possessions, and we can find no fault with her for that, but we should not be outstripped by her in that quarter. We should also have a great canal to connect the lakes with the Mississippi. A very important link in our interior system is also the great canal which begins near us at Buffalo, but it should be made deeper and otherwise improved.

But most important of all to us is a trans-isthmian canal under our own control, to shorten the distance by water from our Atlantic and Gulf coasts to the Pacific. This is no new idea. As far back as 350 years, or soon after the discovery of America, although the Spaniards had an established communication across the isthmus, an easier passage-way was demanded and the project was conceived of constructing an artificial water-way by one of the three routes, Tehuantepec, Nicaragua or Panama, whose relative advantages have been diligently discussed at intervals ever since.

A most important step forward was made in this matter when President Grant appointed a commission in 1872, consisting of several able men, to consider the subject of communication by water between the Atlantic and Pacific oceans. After careful study this commission reported in favor of the Nicaragua route, and nothing has occurred to show that they were not right in their conclusions.

President Hayes, in 1880, presented the subject tersely and forcibly in the following words :

"An interoceanic canal across the American isthmus will essentially change the geographical relations between the Atlantic and Pacific coasts of the United States, and between the United States and the rest of the world. It will be the great ocean thoroughfare between our Atlantic and Pacific shores, and virtually a part of the coast line of the United States. Our merely commercial interest in it is greater than that of all other countries, while its relations to our power and prosperity as a nation, to our means of defence, our unity, peace and safety, are matters of paramount concern to the people of the United States."

A recent writer uses the following strong language :

" But besides the great strategic advantage of the canal, there would be many smaller military advantages given to the United States by the water-way, if, during times of war, it is maintained under the control of the Government. Such, for instance, would be the transport of very heavy ordnance, or torpedoes and bulky material, and of military stores and ammunition from the great manufactories of the East, when the difference of time between sea and land transit would not be important, or when the transcontinental railways are working to their limits. In addition, torpedo boats and other small craft ; monitors, perhaps, or improvised ships ; and possibly very heavy coast-defence vessels with small coal capacity, might pass from ocean to ocean through the canal when it would be impracticable to send them around South America. And, lastly, the armed forces of the United States might be transported from the Atlantic or Gulf to the Pacific without the aid of the railroads in certain contingencies too remote to make it worth while to discuss them here. Such will be some of the advantages of the Nicaragua Canal to the defence.

" But it often happens that the best defence is the attack, and from this point of view a canal will be of exceeding value to the United States. Not only will it permit the speedy passage of troopships to those countries which have in recent years several times shown their hostility to our flag, but it will bring within our reach the merchant shipping of Europe which reaps such a rich harvest along the coast of South America from Chili to Panama, and will enable us to strike great commercial interests like those of the nitrate beds, as well as to reach the war ships and land defences of a western South American State.

" With a power to throw a preponderating force on either seaboard, it is evident that the United States may not only defend her own coasts effectively, but may be enabled to so concentrate her naval strength as to threaten any point north or south on the coast of either ocean or in the West Indies, without weakening too greatly her own coast protection, and thus divert a large force of hostile ships by the necessity of defending such stations as Halifax, Esquimault, Bermuda, Havana or Jamaica; and of guarding merchant shipping in western waters.

" Of course, a canal under American control will divert part of our strength for its defence, but considering the position of the water-way, forming, as it will virtually, a portion of our own coast line, this amounts simply to adding one more port to those already existing on each of the Atlantic and Pacific coasts. Such ports should strengthen rather than weaken the naval defence, since both may be converted by land works into strong harbors of refuge and naval bases, or coaling stations which we greatly need."

It may be considered as certain that the Nicaragua Canal will be

opened before many years have rolled by. If it be not done by the United States, it will be by England or Germany. Let us hope that our legislators will not permit such a thing to happen to us as its control by a foreign power.

It is not unlikely the overland railroads prefer that this water-way be not opened, under the misapprehension that it would injure their traffic, yet everyone who has made a study of the general subject, already ably presented by many, feels assured that the construction of the canal would build up all of our overland railroad interests by developing productions that cannot now have a profitable market, as the railroads cannot afford to carry such bulky and heavy freight at a living rate to the producer.

I will not dwell longer on this highly important feature of our system of national defence, but I take pleasure in calling attention to a most interesting paper concerning it recently printed by Congress. It is from the pen of Captain George P. Scriven, of the Army, and worthy of careful perusal.

I do not claim to be a prophet, but the day seems to me not very distant when the United States will construct and own and hold a ship canal by the Nicaragua route. Then, perhaps, we will extend our domain as far down as that canal and make it a part of our southern coast line. Next, we will absorb Cuba, and then our cup will be sufficiently full.

We have been passing through a period of great commercial and financial depression. But the pendulum never swings continually in the same direction; the reaction will soon come. A revival in business activity will take place; its strides will be rapid and large. Let us hope that a part of it will be the prompt and energetic commencement and completion of the Nicaragua Canal as the work and the property of the United States. The future will surely bring ridicule upon the United States if any other power be permitted to control the Nicaragua Canal.

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A CONCRETE SEWER ON PILES.

By Eugène Lentilhon, Jun. Am. Soc. C. E.

Read June 6th, 1894.

WITH DISCUSSION.

The work described in this paper consisted in the building by the Dock Department, at the foot of Canal Street, North River, New York City, of about 180 ft. of concrete sewer, 16 ft. wide by 7 ft. deep, on a foundation of piles and grillages (Plate LXXIX), with overflow chambers (Plate LXXX) resting on a foundation of piles capped with 12 x 12-in. timbers, and decked. The overflow chambers are designed to carry the storm water and sewage during very high tides. The sewer drains an area of 366 acres, and was originally carried out from the westerly line of West Street in a wooden structure on a crib foundation.

The method of construction from the section *E F* (Plate LXXIX) to the connection inshore with the Department of Public Works sewer was the same as that shown at *E F*. The work was done under the direct supervision of the writer, and in preparing this paper care has been taken to determine accurately the actual cost of construction of each class of work as it progressed, thereby enabling readers to

compare the classified items of construction, with the difficulties encountered and hereinafter described.

The location of the work was such that it was impossible to use any but floating derricks and machines in its construction, and all the concrete, mortar and grout used had to be made in mixing boxes on deck scows, and carried in wheelbarrows along narrow runs from the scows to the work. The oscillation of the scows, due to wind and the swells from passing tugs and steamboats, necessarily made progress slow, and often dangerous, as the runs were about 25 ft. long and from 4 to 8 ft. above the surface of the water. The narrowness of the slip—102 ft.—was a great drawback to the work, as frequently three deck scows 30 x 70 ft., derricks from 30 x 70 ft. to 60 x 70 ft., and a pile-driver 23 x 45 ft., were in use at the same time.

The reason for establishing so low a grade for the bottom of the sewer was that the Dock Department work had to connect with the grades of the Department of Public Works at the westerly line of West Street. The piles were driven by means of a floating pile-driver, the hammer weighing 3 400 lbs., with the exception of about 50 piles driven by a land machine near the westerly line of West Street, where there was not a sufficient depth of water for a floating machine. Cast-iron pile shoes were tried in driving through the cribs of the old sewer foundations, but did not prove successful, and consequently were abandoned. The cost of labor per pile for driving the foundation and test piles, including the rigging of a batter-ways and the land machine, was $3 91, and the actual number of days employed in driving was 65. The prices of the piles follow:

Length of Pile.	Diameter of Pile at Butt.	Diameter of Pile at Point.	Price per Pile.
Feet.	Inches.	Inches.	
40 to 45	12	6	$4 99
45 to 50	15	6	6 85
50 to 55	15	6	8 00
60 to 65	14	7	7 50
60 to 65	14	7	9 50
80 to 85	14	7	10 00

The average cost per pile was $6 07, and over 63% of them were 40 to 45 ft. long.

Several methods of cutting off the piles accurately and economically, for the reception of the grillages at an elevation 1 ft. below

the Dock Department mean low water datum, were tried, and the following method was used and proved successful, the work progressing rapidly and accurately at a minimum cost, without the necessity of waiting for low tides.

A point about mean high water was located on each pile, and as soon as the tide had ebbed to + 1.0, dock-builders got into the water and sewage, frequently being submerged to their necks, and sitting or kneeling on braces temporarily located the day before, measured down with a rod of given length from the point located on the pile, and fastened two small battens on the opposite sides of the pile, to guide the saw. Frequently two men, with a good low tide, have sawn off three piles. Elevations taken on the piles after cutting showed no greater variation than 0.04 ft. Elevations taken on the grillages at numerous points showed a variation from — 0.25 to — 0.33, which could easily be accounted for in the milling of the timber used in their construction. The cost of sawing by this method was $1 28 per pile.

In cutting off the piles for the large foundation blocks at 15.29 ft. below mean low water, a 3-ft. 6-in. circular saw fastened to a shaft temporarily suspended in the ways of a pile-driver, with the necessary belting leading to the engine, etc., was used. These piles, after being sawed off, were located by divers.

The grillage was built in four sections, on a pier adjacent to the work, launched and floated in place at high water and left to ground at low water when possible. This could only be done in the case of the two easterly sections, as an embankment of earth north of the grillage kept washing in over the tops of the piles at each high water; and the flow of the sewer at points west of this was not strong enough to carry away large deposits of sewage, so every pile in this area had to be swept clean before the section could be landed in place. To do this, tides sufficiently low to clean the piles sawed off at a grade of 1.0 ft. below mean low water were necessary, thus causing delay, as the dates of location of the different sections, October 3d, 7th and 25th, 1892, and April 18th, 1893, show. The delay in fastening the last section in place was caused by an unusually severe winter, which prevented the obtaining of cobble stone to place between the piles at the westerly end of the foundation. The cobble stone used in filling up the spaces between the piles was of various sizes, from 6 ins. in greatest dimension down to coarse gravel, and was obtained from a cobble bank

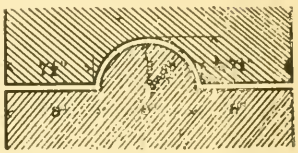
at Cauldwells on the Hudson River, about 45 miles from New York. The scows loaded with this material could not be towed to the work, as the Hudson was closed by ice, and this part of the work was in consequence delayed until the opening of navigation in the spring of 1893. The price of cobble per cubic yard placed in the foundation was 65 cents.

The ten special concrete foundation blocks, of which the five sub-blocks averaged about 63 tons in weight and the five top blocks averaged about 70 tons, set in place by the 100-ton derrick *City of New York*, also caused considerable delay, as the cold weather would not permit the mixing of mortar for the mattresses at the time the pile foundation was ready to receive them. The mattress is used to take up any unevenness in the low-grade piles, and consists of mortar made of slow-setting Portland cement and clean, sharp sand in the proportions of 2½ to 3, the sand screened to ¼ in. largest dimension, placed in bagging sheets and lowered into place by means of a frame swung from the derrick and placed with the assistance of divers. Great inconvenience and some delay was also caused by the narrowness of the slip in setting the blocks. The derrick *City of New York*, 60 x 70 ft., lying alongside of a deck scow, 30 x 70 ft., which carried the concrete blocks, left very little margin for manœuvring the derrick into place after the base block was lifted clear of the deck of the scow.

In all concrete work, broken stone not larger than 2 ins. in diameter, fine sharp sand and Portland cement not less than 400 lbs. gross to the barrel, subject to the Department of Docks' specifications, were used and mixed in the following quantities, called a batch: one barrel of cement, two barrels of sand, and five barrels of broken stone. A batch averages 2 600 lbs. in weight and contains 0.61 cu. yd. The 71 sewer-wall blocks, weighing from 4 to 9½ tons, were transferred from deck scows to the grillage, by 10 or 12-ton floating derricks, as soon as the different sections were finished or the capping decked, and on 3-in. rollers, were rolled into place by hand, with the aid of crow-bars, and a 4 × 10-in. yellow pine lever, 15 ft. long. Great delay was occasioned here also, as the deposits from the sewer during flood tide frequently amounted to 4 ins. in depth, and to give a perfect bearing for the sewer-wall blocks by adzing off of any unevenness of the grillage, it was necessary to wait until the tide had receded to —0.33.

Several weeks frequently elapsed on this account between the setting in place of two adjacent wall blocks.

The sewer walls were made continuous by tongue and groove joints, grouted, as shown in the accompanying sketch of the top of a side-wall block.



The cost of landing, handling and locating the 71 sewer-wall and overflow chamber blocks was $19 39 per block. This work was done by dock-builders. The concrete blocks were built at the East Seventeenth Street Department Yard in wooden moulds, especially erected for the purpose. The cost of concrete per cubic yard in the 10 special foundation blocks, the 71 sewer-wall blocks, and that placed between the I beams, follows :

Class of work.	Cubic yards in place.	Cost per cubic yard for labor.	Cost per cubic yard for material.	Total cost per cubic yard.
10 special foundation blocks.....	234	$4 51	$7 39	$11 90
71 sewer-wall blocks.....	291	5 26	5 94	11 20
Concrete between I beams	212	3 14	5 00	8 14

The beams were telegraphed in place by a floating derrick, located near the established bulkhead line. The telegraph is a primitive but useful method to transfer heavy loads used in dock-building, and consists of a double-drum hoisting engine, a mast or pile-driver ways, some lines, and a snatch block. A heavy line, in this case 5 ins., fastened to some distant point about on a level with the deck of the pile-driver or derrick, as the case may be, is run over a sheave at the top of the pile-driver ways or derrick mast and led to the engine drum. A snatch block is then hung on this line between the points of support, to which the load is fastened with a smaller line, in this case 3½ ins., leading over another sheave next to the first one, then to the second drum of the engine and the telegraph is ready for use. The pile-driver or derrick being anchored, heavy loads can be moved with ease by raising and lowering one or both of these lines.

It was at first intended to place the blue stone on the grillage and decking in a 1-in. bed of mortar (see plates), but tides sufficiently

low to enable the placing of mortar on the grillage when bare were so few and far between that after considerable delay and several vain attempts to have the mortar set in the greasy water issuing from the sewer, it was decided to set the blue stone in place, smooth side down, without mortar.

The cost of building the sewer, its overflow chambers and outlets follows, with the exception of the items of dredging, cobble and rip-rap, piling, etc., and the labor and material accompanying the setting of the base blocks under the oval sewer outlets north of the line marked "present line of concrete blocks" (Plate LXXX).

Cost of making temporary outlet to sewer	$1 041 37
" Piles	3 103 37
" Driving piles, stay-lathing, etc	2 145 03
" Sawing piles off by hand	660 02
" Making and fastening grillages	1 145 63
" 75 787, B. M., ft. of yellow pine timber	1 600 47
" Capping	195 36
" Tree nails and wedges	205 67
" Seventy-one sewer-wall blocks (labor and material)	3 261 14
" Handling and setting sewer-wall blocks	1 377 31
" Bluestone	937 59
" Inspecting, handling and setting bluestone	818 39
" I beams	3 118 66
" Inspecting, placing and painting I beams	1 205 38
" Placing concrete between I beams	665 11
" Ten special foundation blocks (labor and material)	2 647 14
" Cutting and setting granite in five top base blocks	1 251 14
" Setting 10 special foundation blocks	588 72
" Cobble and rip-rap	537 90
" Divers' suits, rubber boots, coal oil, etc	702 88
" Cement, sand, and broken stone	938 21
" Towing scows, rafts, derricks, etc	751 91
" Loading scows with concrete material	167 37
" Excavating for grillage, cutting away old crib, etc	581 48
" Making centers (labor and material)	282 27
" Moving scows and rafting timber	435 44
" Inspecting cobble and rip-rap and laying deck	216 90
" Cutting granite for sewer outlets	84 66
" Watching	1 346 85
Total	$32 013 37

In explanation of the cost of the items for towing, loading scows, rafting timber, and watching, the writer would say that all of the material used, sand, broken stone and cement, was loaded on scows at the West Fifty-seventh Street Department Yard, with the exception of the blocks built at the East Seventeenth Street Yard. All of the sawed timber used was drawn from the West Seventy-fifth Street Department Basin, necessitating, with the transferring of derricks and other floating property, a large amount of towing. As watchmen were required on all floating property, the item for watching will not appear excessive, for watchmen were frequently required at several different places at the same time, to care for the material and property being used for the construction of the sewer.

The engineers, surveyors and draughtsmen's time, and interest on the plant, are not computed in the cost of construction.

The Dock Department rates of labor are as follows:

Occupation.	Rate per Hour.	Rate per Day.	No. of Hours in Day.
Laborer	$0 23	$2 30	10
Foreman of laborers	0.35	3 50	10
Dock-builder	0.30	3 00	10
Foreman of dock-builders	0.35 to 0.45	3 50 to 4 50	10
Engine-man	0.35	3 68	10½
Watchman	0.16⅔	2 00	12
Saw-filer	0.35	3 50	10
Diver	1.00	5 00	4
Diver's tender	0.35	3 50	10
Stonecutter	0.50	4 00	8
Stonemason	0.50	5 00	10

In addition to the above sewer, the standard bulkhead-wall was projected about 35 ft. south of the special sewer outlet base blocks, and a standard department pier 724 ft. long by 40 ft. wide, was built by contract, and carries the sewer out to the pier head line in two oval-shaped creosoted stave barrels 4 ft. 3 ins. x 5 ft. 10 ins. in diameter; 2 760 sq. yds. of granite paving, with the necessary drains and receiving basins, was laid on a 3-in. bed of sand, with coal tar and gravel joints over the area reclaimed, by filling in east of the established bulkhead wall, and a dump, 275 ft. 8 ins. long by 17 ft. wide, was built on the southerly side of the pier, all at an additional expense of about $90 000.

DISCUSSION.

Charles Macdonald, Vice-President Am. Soc. C. E.—I would like to ask Mr. Lentilhon why cast-iron shoes were not successful?

Mr. Lentilhon.—A certain number of pile shoes were ordered and used. I found in some of the work, after we had sawed off the piles for the sewer and got down in the mud around the tops of those along the northerly side of the wall, some of the shoes bent double. In many instances where we had to drive through rip-rap, we used this same sort of shoe with great success.

Mr. Macdonald.—I understand it is not your opinion that the cast-iron shoes were necessarily unsuccessful in such work?

Mr. Lentilhon.—It is not my opinion; they were not really abandoned.

Mr. Macdonald.—I notice in the table, on page 573, you give the cost per cubic yard of material for special foundation blocks as $7 39, and in the concrete for the eye-beams as $5; what was the reason of that difference?

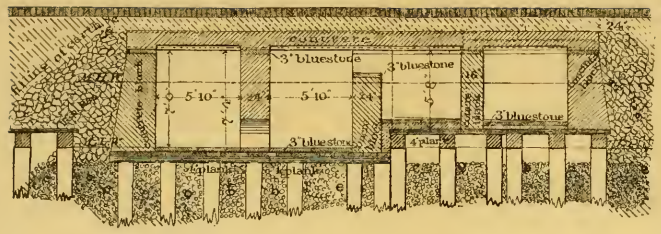
Mr. Lentilhon.—That can be accounted for by the fact that entirely separate molds of peculiar shape had to be built for those blocks; whereas, for the concrete put in the eye-beams there were no molds at all except the planks at the north and south side of the beams. We picked up any old plank we could find for that.

Mr. Macdonald.—Is not even $5 a rather excessive charge for materials—for concrete? Was there any reason why your material should have cost $5?

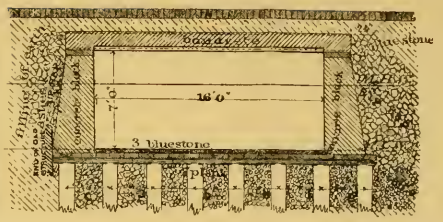
Mr. Lentilhon.—At that time, I believe, the cost of Portland cement was $2 04 to $2 20 a barrel; it averaged about $2 14. The same cement to-day is only costing $1 80. It took one barrel of cement, with the broken stone and sand, to make $\frac{6}{10}$ cu. yd. of cement. It may be interesting to state here, in connection with the ten special foundation blocks which cost $11 90 a cubic yard, that the records of the Dock Department show that between 1890 and 1893, 329 such blocks in all were made, and if we take an average weight of about 65 tons per block, it would make 21 385 tons, of which the average cost was $9 59 a cubic yard.

A. A. Stuart, M. Am. Soc. C. E.—I desire to ask the author why concrete was used in building those walls in lieu of stone masonry. As I understand the figures given in the paper, the blocks used in the walls cost about $12 per cubic yard, exclusive of setting, and if this understanding is correct, it would seem that concrete was used because of its supposed greater durability than good ashlar masonry, which in most localities can, in similar situations, be built for no greater cost than that given as the cost of this concrete. My under-

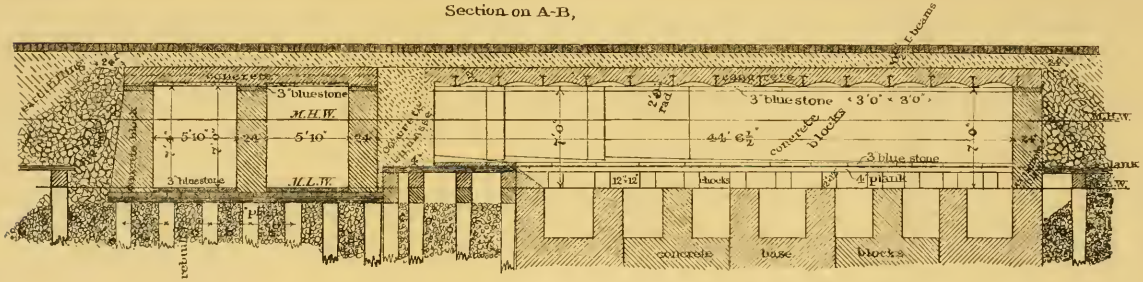
Section on C–D,

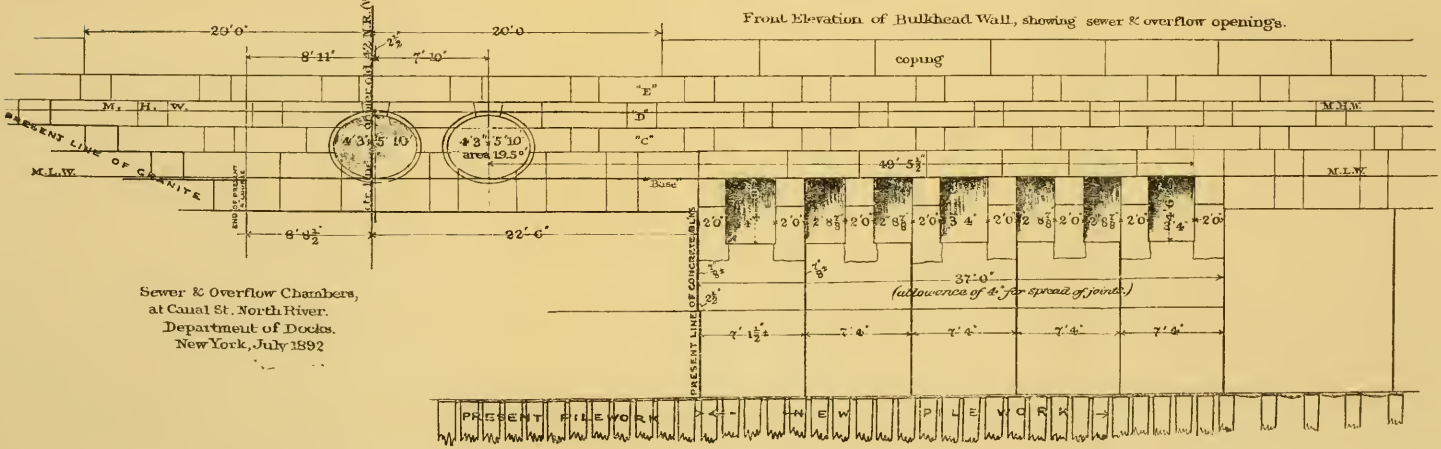


Section on E–F.

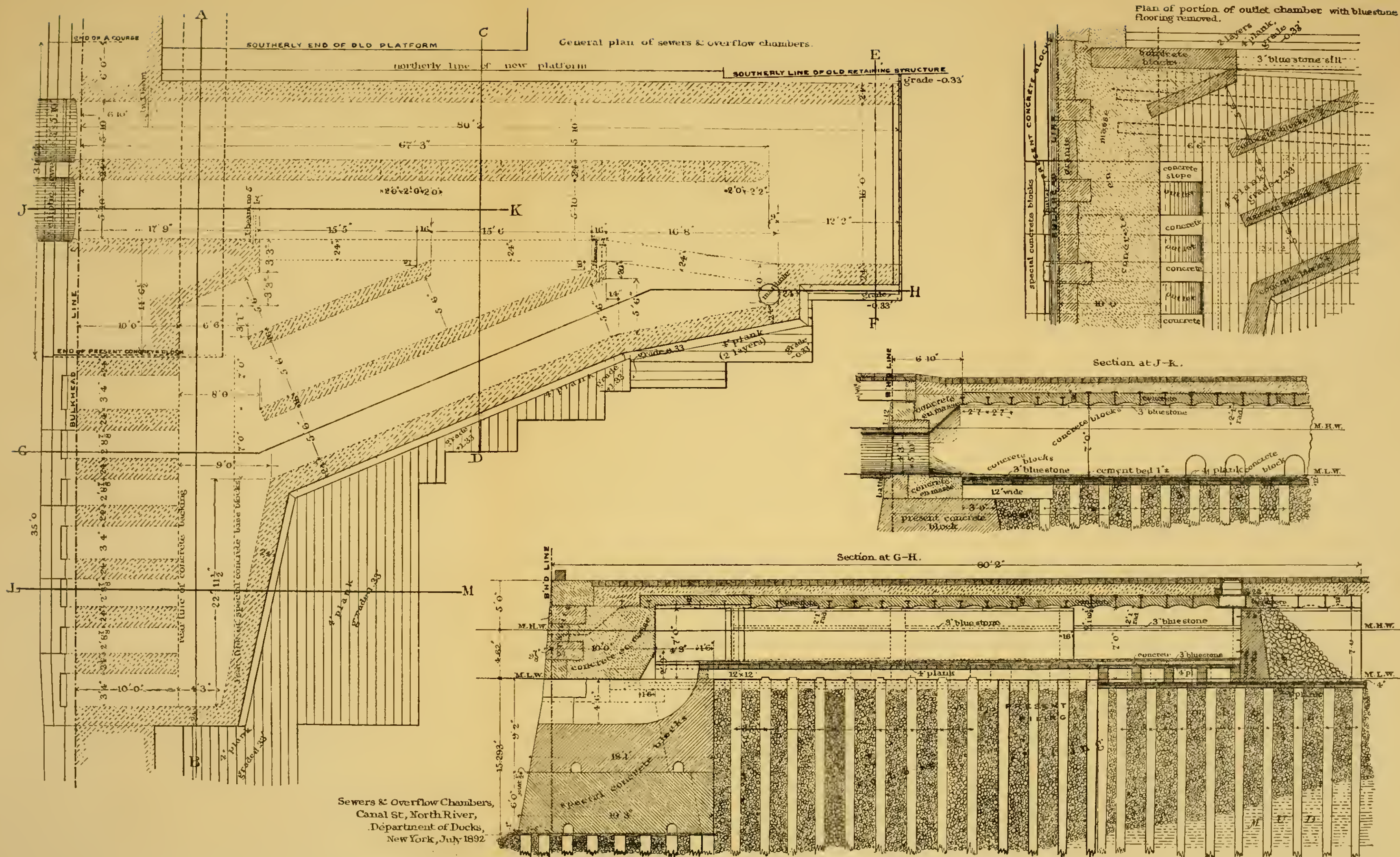


Section on A–B,





Sewer & Overflow Chambers,
at Canal St. North River.
Department of Docks.
New York, July 1892


General plan of sewers & overflow chambers.
SOUTHERLY END OF OLD PLATFORM
northerly line of new platform
SOUTHERLY LINE OF OLD RETAINING STRUCTURE
END OF PRESENT CONCRETE BLOCK
BULKHEAD LINE
Plan of portion of outlet chamber with bluestone flooring removed.
Section at J-K.
Section at G-H.
M.H.W.
M.L.W.
Sewers & Overflow Chambers,
Canal St, North River,
Department of Docks,
New York, July 1892

standing has been that concrete is used in building sea walls and similarly exposed structures, because of the fact that it can be made into larger individual blocks than can be procured from the average quarry at moderate cost, but as the blocks used in this work were not beyond the range of most quarries to furnish at reasonable cost, it appears that considerations other than that of economy must have suggested the use of concrete. If it is thought to be more durable than good masonry, the statement of that fact would be an interesting one to have brought out.

Mr. Lentilhon.—I would say that opens up a very interesting subject for discussion which I am not here to discuss or criticise. That is Mr. Greene's bulkhead wall; it has proved a success.

Mr. Macdonald.—Gentlemen who are familiar with the bulkhead wall will remember that it was necessary to place large masses in position, and the only convenient and inexpensive way in which to obtain monoliths of this size was by forming them in molds in concrete. That was determined upon early in the construction of the dock wall. I am sorry that Mr. Greene himself is not here; he could, perhaps, enlighten us. I recollect hearing the matter discussed when General McClellan was Chief Engineer of the Dock Department and Mr. Van Buren was his principal assistant.

Wm. Barclay Parsons, M. Am. Soc. C. E. (by letter).—The paper by Mr. Eugene Lentilhon has the merit, in addition to being interesting, of setting forth in detail the cost of the work. But it is not quite clear why there is the great difference in the cost of concrete in the foundation and wall blocks and that placed between the I beams, where the cost of the materials varies from $5 to $7 39 per cubic yard, although he states that the mixture was the same in all cases. Neither is it clear why the labor for the concrete between the beams is so very much less than the cost of the labor for the concrete in the blocks, since, in the first place, the amount of concrete was small; whereas, in the other case it was used in large masses, and therefore should have been much more cheaply handled. The total cost of the concrete per cubic yard in foundation and wall blocks seems very high, especially as these blocks were very large, thus giving an opportunity for cheap work per cubic yard. According to the paper, the item $11 39 is the cost of the concrete blocks in the moulds, because there is a separate item for handling and setting the same.

The total cost of building 180 ft. of sewer may seem very high, but it must be remembered that it was built through very bad ground, and was built according to the New York Dock Department standards, where excellence of workmanship and material are the first considerations. The care with which the work is done is shown in the small item of watching, which amounts in this case to over 4% of the whole, a percentage usually enough to cover the expenses of engineering;

whereas, in this case, the watching applies solely to the day and night men employed to take care of the plant and material.

Mr. Lentilhon.—The point of the difference in cost between the concrete put between the I-beams and that in the moulds has already been brought up. The reason for the cost per cubic yard being greater is, that in making the concrete in the first case, laborers at 23 cents an hour were employed. In making the concrete in blocks, in addition to the laborers actually employed in mixing the concrete, it takes carpenters and expert mechanics, who received 39 cents an hour, to make the moulds, which cost is figured in the cost per cubic yard of concrete.

In making five special foundation blocks there were two molds of one kind and three of another. The reason for giving an average for so many wall blocks, covering the period between 1890 and 1893, was to illustrate the difference in price of materials, labor, etc., as used in the wall and that in the sewer. In that period there were probably not more than half a dozen molds made—there might have been more; and they were used until they wore out. The difference in cost between the standard wall blocks and the sewer-wall blocks was about $2 a yard.

Mr. Macdonald.—Is there anything, in your judgment, that would prevent doing work of that kind by contract, under carefully drawn specifications? I ask, because the law, as I understand it, has now been amended so that all work in the Dock Department must be done by contract. In your experience did you form any opinion as to whether it would be practicable to insure good work by contract under such circumstances?

Mr. Lentilhon.—Yes, sir; particularly in concrete work, the foremen that have been with us for years have to be urged up to their work all the time. For various reasons we have to keep shoving our men around, particularly in ramming the concrete wall-backing in low-water work; the foremen frequently fill in the concrete too hastily and try to do too much work in one day for a record. If we had Italian laborers doing the work, with contractors, I have my doubts whether the work would be done as well as now.

Mr. Macdonald.—It is a question that the taxpayers of New York are especially interested in, not only the Dock Department, but the Park Department and the other branches of construction, as to whether work can be done by the day enough better than it can be done by contract, under carefully prepared specifications, to warrant the day system in preference to that by contract. The Legislature seems to think that all work should be done, in municipalities like New York, by contract. The engineers who are employed by such municipalities must make up their minds to devise some scheme of inspection whereby the wishes of the Legislature shall be carried out and good work obtained.

A Member.—Why not extend the contract system to the employment of the inspectors and the supervision of the work? Let that be

done cheaper, too. If the law is good so far, why not extend it as far as it will go?

Mr. Macdonald.—It is not a theory, but a condition that we have to consider. The fact is, the law is as I have stated it. It is a question whether engineers are going to be able to show themselves competent to conform to the law or not.

W. W. Kenly, M. Am. Soc. C. E.—Mr. Chairman, there is one question I would like to ask, whether the concrete work was done entirely by hand labor or by a concrete mixer ?

Mr. Lentilhon.—I will state that the Dock Department does not employ mechanical assistance at all in their concrete work.

Mr. Kenly.—Is that a question of politics or is it a question of making it better by hand than by machinery ?

Mr. Lentilhon.—I guess it combines a little of both.

E. B. Gosling, Jun. Am. Soc. C. E.—I was connected formerly with the Department of Docks, and at that time had under advisement the putting up of a plant for mixing concrete by machinery. We found we had not spare space enough along the water front to prepare and store the number of blocks that economy would require us to make. Besides we would have been obliged to strengthen considerably the wooden bulkheads, to enable them to support safely the additional loading; consequently the plan was not adopted. I had suggested a machine with two inclined helices, which revolve in opposite directions and give a very thorough mixing of the materials. The machine is rather expensive, and was the invention of Mr. Coignet.

Mr. Kenly.—Inquiry was made as to the relative efficiency of contract work and day labor in the public works of the city. I have had a great deal of experience, in public work especially, and my experience has been there is no great difficulty in getting good work done by contract. As to whether you can get good work done just as well under the engineer as by day's work, I think that opens up a question of very serious importance to the Society. In my experience you can get just as good work by contract as by day's work. It is a question that involves the character of the engineer, and I do not see why, with a competent engineer with sufficient experience, the work cannot be done just as well. On large railroad works it is done in that way. Take the construction of the Baltimore system of water works, built for $125 000 less than the appropriation; that was a remarkable piece of work.

Mr. Macdonald.—Which shows that the engineer had a wise conception of the problem before him in making his estimates.

Mr. Kenly.—He also had a wise conception of the interests of the tax-payers. But that is a question that I think is very vital to the interests of the profession of civil engineering. The statement was made that you cannot get as good work under contract work as under day's work. I think that is rather an impeachment of the ability and character of the engineer.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

712.

(Vol. XXXI.—June, 1894.)

FAILURE OF A MASONRY PIER AND A ROCK FOUNDATION.

By WM. BARCLAY PARSONS, M. Am. Soc. C. E.

READ MAY 16TH, 1894.

WITH DISCUSSION.

On the line of the Central New York and Western Railroad, 15 miles east from Hornellsville, N. Y., there is an iron viaduct 700 ft. long, with a maximum height from stream to base of rail of 235 ft. This structure, the highest in the State of New York, was erected in 1883. The tallest tower has two of its legs resting on piers which stand in the bed of the stream. About five years ago these piers showed signs of failure and were encased in a jacket of concrete 12 ins. thick. Cracks, however, soon appeared in the concrete, which were stopped with cement from time to time, but always reappeared.

In the autumn of 1893, the superintendent of the road being anxious about the piers, the author was called in by the company to make an investigation. The concrete of one pier was found to be very badly cracked in several places. One crack, 4 ins. wide, extended from top to bottom, a height of 10 ft. The other pier was considerably, but not so

badly, damaged. The bottom and sides of the glen, the latter being very steep, consisted of a shale rock in alternate layers of hard and soft material, as will be seen from the photographic illustration accompanying this paper.

When the concrete coating was put on the piers it was founded on the top surface of the shale as exposed. The action of water, air and frost soon produced their well-known decomposing action, and the shale was so badly eaten out from under the concrete as to produce settlement and cracks. The author attempted to save the pier and concrete by excavating a trench in the rock around the work to be filled with concrete, and so stop further decomposition of the foundations; and then he proposed to fill all the cracks with strong cement mortar. But the jar produced by excavating the shale brought down a large piece of the concrete jacket, exposing the pier itself, which was at once seen to be in such a rotten condition as to be beyond salvation.

On removing the whole of the concrete, the pier was found as shown in Plate LXXXI. The pier originally had been built directly on the exposed top of the shale without any excavation, and the decomposition of the shale had wrecked the pier itself in the same manner as it afterwards destroyed the concrete jacket. An examination of the plate will show that previous to the putting on of the concrete an attempt had been made to take up the loss resulting from the decomposition of the shale by shoving beneath the masonry small stones and gravel. Attention is called to the pedestal stone, which was not only badly broken, but actually dished, which the photograph exhibits very distinctly. The stones composing the masonry were a poor quality of sandstone, breaking readily, and in this respect the plate fails to do justice to the badness of the masonry, because every stone was more or less cracked.

In order to support the tower during the removal of the pier, the false work, as shown in Plate LXXXII, was devised. This consisted of two small plate girders 5 ft. long and 30 ins. deep, with web plates $\frac{3}{8}$ in. thick, which were fastened with turned bolts to the leg of the viaduct. This leg, which carried one-half of a 30-ft. and one-half of a 100-ft. span, consisted of three 12¼-in. channels. The projecting ends of the plate girders rested on four 20-in. I beams, 240 lbs. per yard. These beams were 18 ft. long and were supported at their ends as shown by two cross-timbers, resting on three 30-ton hydraulic jacks at each end. Two stout cribs of old-bridge timbers were built up to receive

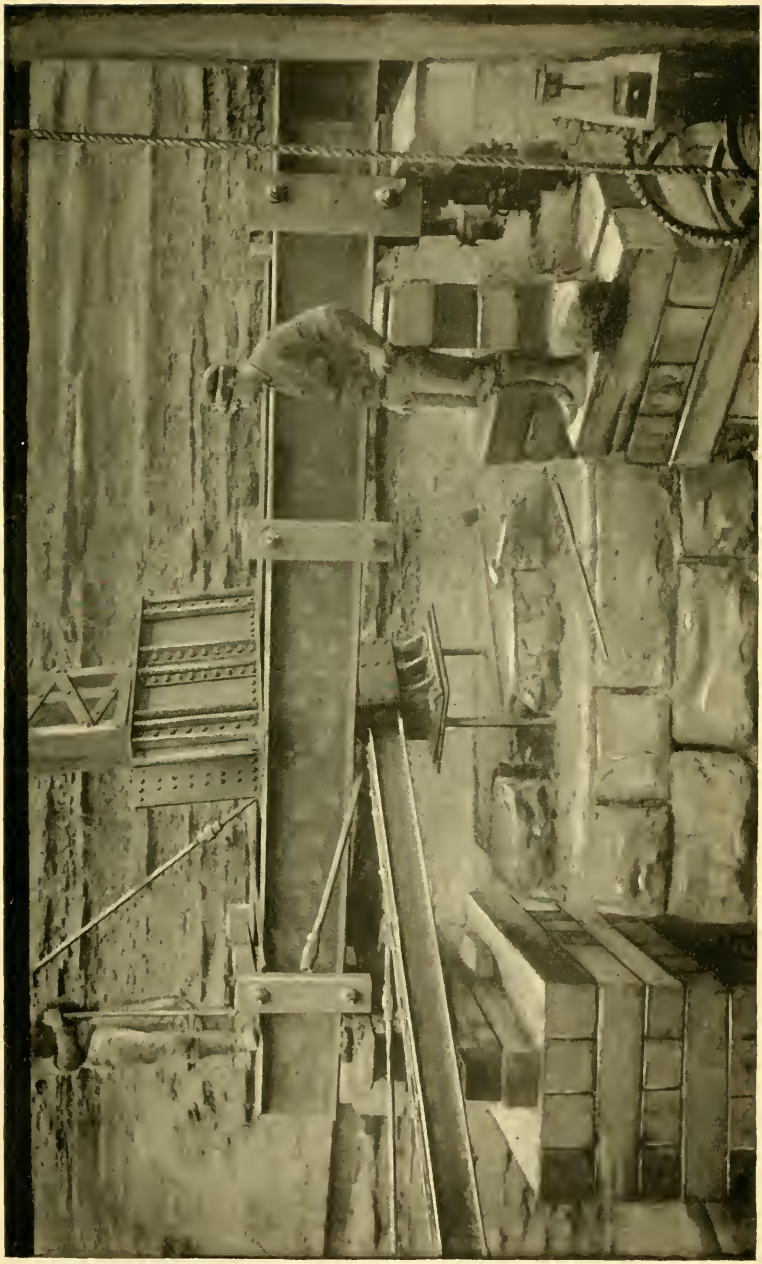
the jacks. The up-stream crib was made V-shaped in order to provide a breakwater for the stream in case it should rise during the progress of the work, and also to permit of inserting the blocking, so as to shorten the effective span of the supporting I beams. It was well that this was done, because the stream rose suddenly, and had there been a square crib, damage to the work would probably have resulted.

The jacks lifted the iron work clear of the old piers, when the blocking was immediately introduced, so that the beams rested on the blocks and jacks. The old pier was then readily removed. As was expected, the masonry was found to be in a very bad condition.

The large stones were but little more than a veneer for a dry center. The vertical face joints were in many cases but an inch deep, the stones being triangularly shaped with the bases outward. The masonry was easily removed with a bar and shovel.

Before putting up a new pier the shale rock was excavated to a depth of about 3 ft. and to a similar distance outside of the lines of the new pier. The new masonry was made with Leroy limestone, a very hard but durable stone. The stones were carefully cut with full joints, so that very little backing was used. The concrete mixture in the foundation consisted of 1 part of American Portland cement, 2 parts sand, and 4 parts broken stone. In laying the new masonry, the space beneath the iron being quite free of staging, the work proceeded in the ordinary manner, except as to placing the pedestal stone. It was evident it could not be set by dropping it over the top of the holding-down bolts. It was therefore put in place first and the long bolts run through it from below, the stone being held against the iron base plate. The pier was then built up to it, the coping stones having been carefully laid out so that the bolts passed through the joints. The base plate of the leg having been slightly bent, owing to the breaking of the original pedestal stone, a piece of sheet lead was inserted between it and the new stone in order to ensure an even bearing.

The cost of doing the work, all expenses included, was $1 041 92. Mr. G. R. Tuska, Jun. Am. Soc. C. E., had immediate charge of the work as Resident Engineer.



DISCUSSION.

J. Foster Flagg, M. Am. Soc. C. E.—I would like to ask Mr. Parsons if, during the erection of the new piers, the superstructure was held up in part by hydraulic jacks? I should not suppose that hydraulic jacks could be depended upon to carry such a load for any length of time without settlement, from the unavoidable leakage of the fluid which operates them. I should think that the load must have been practically all carried by the blocking.

Bernard R. Green, M. Am. Soc. C. E.—I think that the same lack of engineering advice must have existed when the concrete casing was put on the old pier only 12 ins. thick. It seems to have been a ridiculous method of protecting the old pier. I take it that the concrete was not applied merely to protect the pier from the effects of the atmosphere, but as a hooping or encasement, relying on the tensile strength of it to hold the pier together by resisting a bursting tendency. The thermal expansion and contraction of so thin a shell of concrete, exposed to the weather, would tend to crack it, through the natural action of the loose material inside. Small stones or particles of sand, etc., falling between the casing and the pier, after expansion in hot weather, would remain there as a fulcrum or chock when contraction followed. Cracks would be certain to result in such a case.

I will show on the blackboard a peculiar effect of expansion and contraction that occurred in the granolithic pavement around the Washington National Monument some years ago. The base of the structure is 55 ft. square and the granolithic pavement or apron about it, intended to catch the wash of rains running down the shaft, as well as to serve as a smooth foot walk about the monument, is 10 ft. wide. This pavement was therefore a hollow square inclosing the base of the shaft, about 8 ins. thick, and bounded by a granolithic curb. Of course, when under the heat of a Washington summer sun, expansion took place in all directions. It pushed all the corners outward diagonally, so that the pavement took the form of a slightly concave-sided square. The middle parts of the inner sides of the square remained in contact with the monument, but the corners were ½ in. off, and the cracks tapered to nothing at the middle points. Naturally when the pavement contracted again a little crack occurred squarely across it each way from each inner angle, and when it expanded again the dust and dirt accumulated in the cracks prevented closing and even operated to push the corners of the pavement still farther outward.

We all know that coping stones will not lie still on a wall. They are always moving from expansion and contraction, and unless tied together with something that will draw them up again when contrac-

tion takes place, permanent cracks and displacements must result. I think that was the cause of the cracking in the casing of the pier, and that the latter could not have amounted to much as a reinforcement.

James Owen, M. Am. Soc. C. E.—I have used concrete packing with success where I have put piers on shale. I have found that exposure to the atmosphere would have a tendency to disintegrate the shale, and I have protected them with concrete and found it efficacious. There was no weight on the concrete itself, of course. The peculiarity of some shales is that mere exposure to the atmosphere will disintegrate them while a protection with a covering of that kind will prevent further inroads in the material itself.

P. F. Brendlinger, M. Am. Soc. C. E.—I am glad Mr. Parsons is here; perhaps he can give us something that the paper lacks; that is, the history of the original construction. Apparently a contractor ran the work. Now that contractor was not an engineer, and for that reason, I say, and have said many times, that engineers should study and practice both engineering and contracting as much as possible. While it will not detract from the quality of the engineer, I think it will elevate the quality of the contractor. Here is a piece of work that, if the company had any regard at all for its money or for its future traffic or prosperity, should have been looked after. If a contractor knows his work is inspected, there is never such work as this going on. An old engineer said to me 20 or more years ago, "Any one can build a pier, but it takes brains to build a foundation." Make your foundation secure and it does not make so much difference—although, of course, it is essential to have it good—about your masonry, and your structure will still stand. No matter whether I had charge of 8 miles of road or 100 miles, I always made it a practice to see every foundation before work was started on the masonry. I never had but one case in my life in which masonry gave way that I had anything to do with, and this case was not a foundation, but the top of an abutment.

The abutment that I have reference to is the north abutment of the double-track bridge crossing over the Mount Carbon and Port Carbon Railroad, at Pottsville, Pa. The span is about 200 ft. The weight of the iron work alone is about 150 tons on each abutment, or 75 tons on a shoe, under each end post.

The foundation was 14 ft. 4 ins. deep, and 11 ft. thick. The masonry of the abutment, above ground, was 10 ft. wide at the surface, and 7 ft. under the bridge seat; 19 ft. high. The face of the masonry was vertical, so as to bring the abutment as near to the railroad crossed as possible, to reduce the span. The body of the masonry was built of Hummelstown brown sandstone, a very brittle stone, while the bridge seat was built of Conshohocken stone, a species of trap-rock, which is considered *par excellence* for bridge masonry, and especially bridge seats.

The bridge seat, in this case, was 5 ft. deep, 18 ins. thick.

The contractor had been paid in full for this work, the bridge was erected and the road was in the hands of the operating department. In the course of a few months I noticed that the stone on which one end post rested was slightly cracked. In a week or so the crack had become larger, and the stones of the abutment were cracked for three courses below the bridge seat. I knew the foundation must be good or else the cracks would have started at the bottom and extended upwards. The cracks did not extend below the third course from the top. So it was decided to tear out the cracked masonry. The bridge was carefully propped up by the operating department and the contractor who built the masonry tore down the cracked part and rebuilt the same with Conshohocken stone throughout (at his own expense). The cause of the cracking was held to be some under-pinning with chips, but it was an open question with me whether the brittle Hummelstown sandstone was not brought too near a 75-ton pressure, in a small area.

F. Collingwood, M. Am. Soc. C. E.—A partial failure, similar in kind to that mentioned by Mr. Brendlinger, occurred in one of the piers of the Allegheny Suspension Bridge at Pittsburg. I was called upon in 1883 to examine the bridge, and make such repairs as were needed. I found that when the suspension bridge was built, there had been a widening of the spans; and one of the piers had been enlarged by building new work at the side and ends and across the top of one of the old ones. There was a vertical crack in this pier some 10 ft. long, under one of the iron columns supporting the cables, and some of the stones at other points had cracked more or less and weathered badly.

It was not considered safe to remove more than one stone at a time. When this was done the work in the interior of the old pier was found to be full of spaces, the grouting used not having penetrated throughout the mass. Grout was run in wherever possible until no more would be received. The new stones from a quarry, known to be of good quality and cut to $\frac{1}{2}$-in. joints all round, were then slid into place on a bed of rich cement mortar. The top bed joint was then filled by stiff mortar thoroughly rammed in, and the side joints filled by the use of a sword in the usual way. No settlement occurred and no further cracking has been observed.

Mr. Parsons.—In reply to Mr. Flagg, the superstructure was held up during the lifting process entirely by the hydraulic jacks, but after it was lifted clear of the masonry, blocking, as shown by the illustration, was introduced, and the jacks were set up from time to time, so that the load of the superstructure and of the trains was carried by both blocking and jacks; either, however, would have been sufficient to do the work.

As to the original structure, I think there was a decided lack of good engineering advice, and when the pier was encased after it had commenced to fail, it was done by the superintendent of the road without, I believe, any engineering direction.

Originally the builders were entirely misled by the appearance of the shale rock, which looked hard, and was hard, but which disintegrated in the course of time.

The second pier has not yet been repaired, the concrete jacket still being in place. The work described was commenced in November and ended in December. We did not care to undertake any more work at that season of the year than was absolutely necessary. The cracks in the second pier were seen, however, to be increasing. I satisfied myself that this was due to the rain water freezing and thawing, as the weather was wet and alternately cold and warm. I had all the cracks very carefully gone over, cut out, filled with cement, and carefully covered up so as to prevent any rain water from running in. This stopped the deterioration of the jacket during the winter. It was, however, but a temporary expedient, and the pier will soon have to be rebuilt.

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713.

(Vol. XXXI.—June, 1894.)

THE RECONSTRUCTION OF A PORTION OF THE SUBSTRUCTURE OF THE JOHNSONVILLE BRIDGE.

By WALTER H. GAHAGAN, Assoc. M. Am. Soc. C. E.

READ MAY 16TH, 1894.

WITH DISCUSSION.

The writer recently finished, in the capacity of Superintendent of Construction, the removal of three old piers and the building of two new ones, for a portion of the Johnsonville Bridge. This bridge is the crossing of the Nashville, Chattanooga and St. Louis Railway over the Tennessee River at Johnsonville, Tenn., about 80 miles west of Nashville.

In March, 1893, Hunter McDonald, M. Am. Soc. C. E., Chief Engineer of that road, asked for proposals for the construction of a new draw bridge at Johnsonville. This paper will present a description of the old piers and some of the interesting features of the work of constructing the new.

The old piers were built in courses varying from 12 to 30 ins. in thickness. The face stone had been carefully cut and laid with headers and stretchers, and the backing filled in with rough pieces of stone

which would be classed as large and small rip-rap. The stone had been laid without mortar of any kind, and the piers seem to have stood remarkably well. In the lower portion of the piers the high water had left silt and sediment until all the voids in the masonry were entirely filled. In the upper portion of the piers there had been poured, in late years, cement mortar grout which filled some of the voids and assisted to bind this part of the piers together. All of the piers had large cracks in them, caused by unequal settlement of the foundations. There had also been a settlement of one of the piers of about 6 ins. more on the down-stream end than on the up-stream end. They were also considerably out of plumb in the other direction.

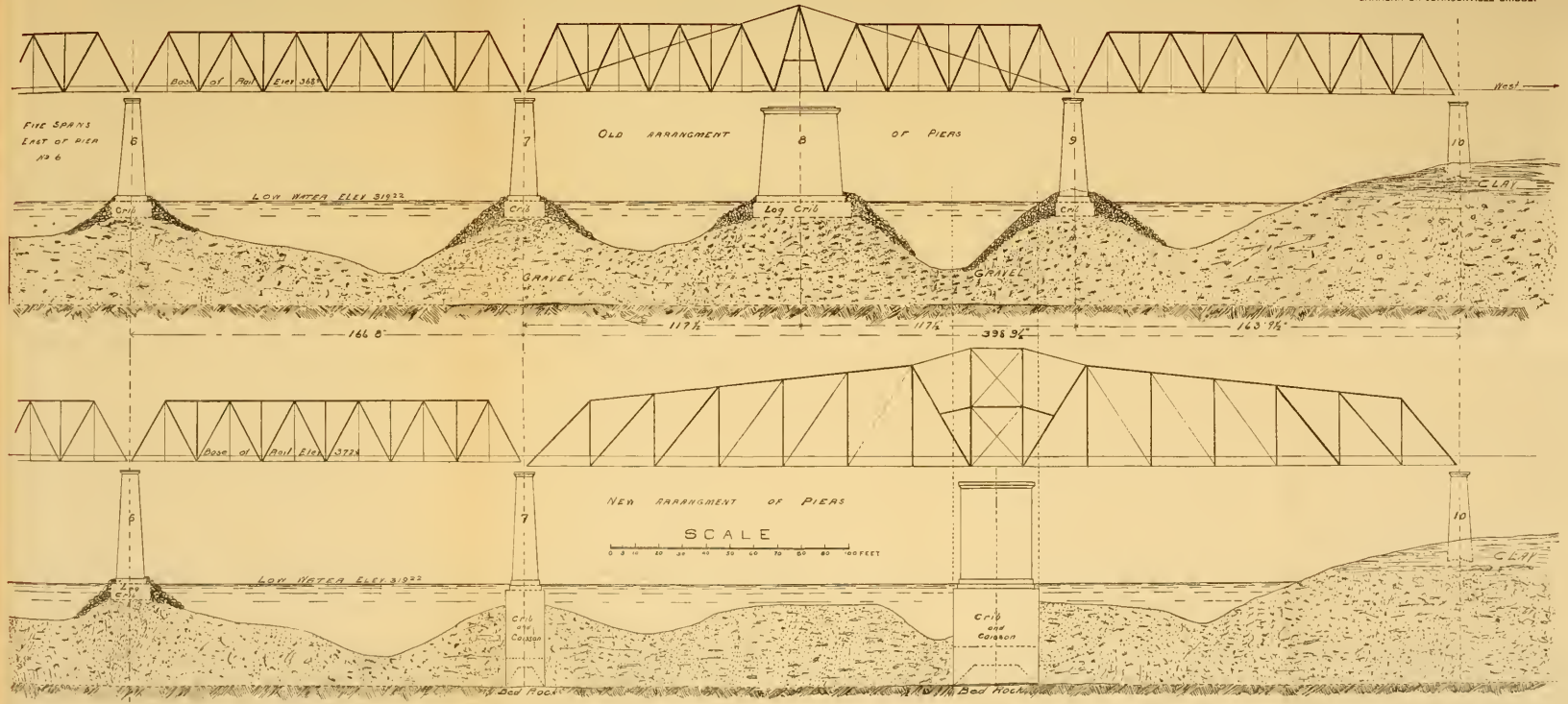
These old piers were founded about at low water upon rough timber cribs which rested upon the gravel of the river-bed. Upon removing these old cribs we found them to be built from 6 to 7 ft. high, from round logs of poplar and oak timber. The logs were sized one way and halved at the ends. Round drift bolts, $\frac{5}{8}$ in. diameter, had been used to fasten them together. These cribs had been filled with rip-rap stone. Very large stone, having an area of 8 to 16 sq. ft., were laid in a course near the top. Most of this top course of rip-rap stone, as well as those in the footing course of the piers, were found to be broken from unequal pressure when they were removed.

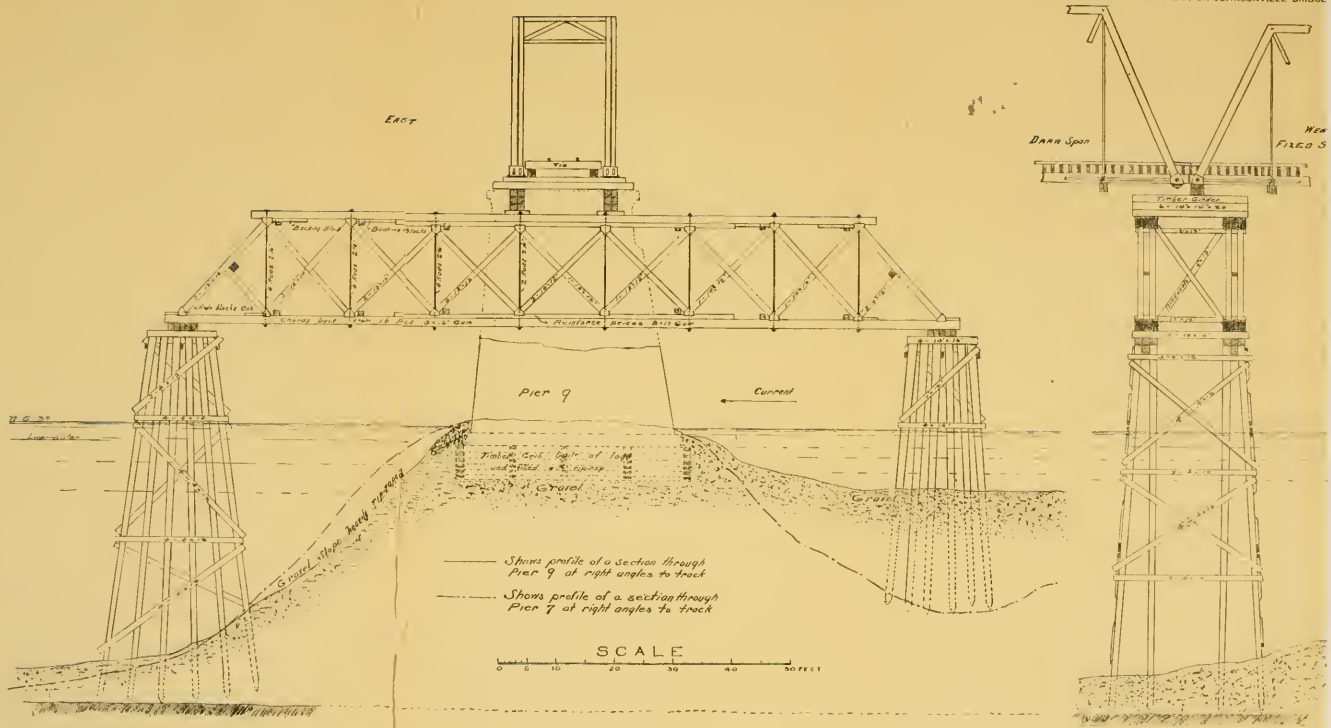
The old piers, 7, 8 and 9, as shown on Plate LXXXIII, were torn down. The foundation cribs, and all material surrounding them, were removed to a depth of 8 ft. below low water. As will be seen by Plate LXXXIII, the old cribs were set upon the gravel bed of the river and were protected by dumping rip-rap around them. This materially reduced the section of the river and caused scour. Additional rip-rap was thrown in for protection as the scour increased, until the scour reached to within a few feet of bed rock below the piers.

By examining the profile sections on Plates LXXXIII and LXXXIV, it will be seen that the piers rested upon a mound of gravel protected by rip-rap. The depositing of this rip-rap around the piers and in the already narrow openings was beginning to interfere with the passage of boats through the draw. It was necessary, during the construction of the new piers, to permit the draw span to swing at any time for the passage of large and small boats, which averaged between one and two boats each day.

In the invitation for proposals, furnished by Mr. Hunter McDonald,

PLATE LXXXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 713.
GAHAGAN ON JOHNSONVILLE BRIDGE.




EAST
Pier 9
Current
Gravel
Gravel
Shows profile of a section through Pier 9 at right angles to track
Shows profile of a section through Pier 7 at right angles to track
SCALE
0 5 10 20 30 40 50 FEET
Draw Span
Fixed S

it was required that a new pier be erected directly in place of old pier 7, and a new pivot pier at a location shown on Plate LXXXIII. Also, required to support old spans 6–7, 7–8, 8–9 and 9–10, in such a manner as to admit of the safe passage of all trains and steamboats until the work was completed. The proposals suggested a method of supporting the old spans while the new piers were being constructed. Two trusses, 60 ft. center to center of end pins, would have been furnished to the contractor by the railroad company. The pneumatic method of sinking was preferred, but other methods would be considered. Also, the contractor was required to furnish a complete description of the method to be used in sinking foundations and supporting the old spans, accompanied by drawings and specifications.

The substructure work was awarded to the H. S. Hopkins Bridge Company, of St. Louis, Mo. Mr. Frank D. Moore, C. E., Vice-President of that company, designed and furnished the working drawings for two caissons and the two trusses and pile piers shown on Plate LXXXIV.

The H. S. Hopkins Bridge Company then made an arrangement with McGee, Kahmann & Co., of Kansas City, Mo., by which all of the work agreed to be done by the former party should be superintended and performed by the latter.

Supporting the Old Spans.—At one time it was the intention to adopt the method of supporting the old spans as shown on Plate LXXXIV, at pier 7 only. This method consisted in erecting two heavy trusses, one each side of the pier to be removed, and at right angles to the track. These trusses to be supported by pile piers, these pile piers to be driven far enough from the old piers so that the driving would not disturb the slopes of the material upon which the old pier rested. Also, to have these pile piers far enough away from the site of the caisson to be sunk, so as not to be disturbed by this sinking; these trusses to be braced together. Blocking to go on the top chords to support the ends of the old spans. The end for the draw span being arranged so it could swing and be latched with no more trouble than when it rested upon the old piers.

At old pier 9 it was attempted to use the old method of driving piles each side of the pier and near to it. Upon these piles timber bents would be erected, which would support the ends of the spans while the new pivot pier was being sunk. In June the work of driving

these piles for this second plan was commenced at the west side of old pier 9. After three or four piles had been driven, the top of the old pier had settled 4 ins. toward the west. After this experiment it was not considered safe to do any more pile-driving near or around old piers 7 or 9 which would disturb the slopes of the mounds upon which these piers rested. This latter plan was then abandoned, and it was decided to use the two trusses at old pier 9 first, while the pivot-pier caisson was being sunk, then remove them to pier 7, for use there.

On July 1st the writer was placed in charge of the work by the contractors as Superintendent of Construction. At this time most of the material for the trusses and the caissons was still uncut by the mills. All of the material for the trusses was green and unseasoned; none suitable could be gotten in any other condition. The chords of the trusses were made from 16 lines of 3 x 16-in. gum timber, in lengths from 17 to 43 ft. reinforced by 8 x 12-in. gum timber. This made the cross-section of the chord at its middle 4 ft. wide and 2 ft. deep. The vibration braces were of 12 x 12-in. and 8 x 12-in. materials. The top and bottom laterals were of 4 x 12-in. materials. The angle blocks were framed from a piece of green oak timber 16 x 20 ins. x 4 ft. Each truss was divided into 9 panels, 14 ft. 6 ins. each. The vertical distance between the centers of the angle blocks of the top and bottom chords was 16 ft. 6 ins. for the end panels, and 15 ft. 2 ins. for the four middle panels. The trusses were placed 15 ft. center to center, and were 130 ft. 6 ins. from center to center of end angle blocks, or 136 ft. 6 ins. over all.

These trusses were framed and erected before any of the timber had time to season. The seasoning took place after the load was on the trusses and made it necessary to give them a good deal of attention in tightening up bolts and taking up on the vertical rods. As the timber seasoned and shrunk the angle blocks reduced in size from $\frac{1}{2}$ to $\frac{1}{4}$ in. With the dead load of the old spans, and trains passing every few hours, we could not take this shrinkage up by screwing up on the nuts of the vertical rods, and the trusses sagged so the center was $5\frac{3}{4}$ ins. lower than the ends before the timber became seasoned. Part of this settlement may have been due to the fact that the members were all heavy and large, and a perfect bearing was not obtained until the load came upon the trusses. The writer believes, however, that it was nearly all due to the seasoning and shrinking of the timber.

These trusses remained at old pier 9 until after the first pivot caisson had been sunk to the bed rock and the working chamber sealed. Old pier 9 had been removed in the meantime.

Sixteen piles were then driven from the track on the bridge between the trusses forming a pier for the support of the ends of the spans, so that the trusses could be taken down and re-erected at pier 7. In removing these trusses the top and bottom chords were not unpacked; they were picked up bodily and lowered into the water, then floated to pier 7, where they were picked up again and put in place when the trusses were re-erected. One bottom chord piece weighed 44 000 lbs. and was 108 ft. long as it was lifted into place at pier 7.

When the trusses were re-erected they were shortened one panel on each end, making seven panels of 14 ft. 6 ins. instead of nine panels. This change was made because the pile pier on the down-stream side was more exposed at pier 7 than at pier 9, and more likely to be run into by passing steamboats. It was desired to have more material around the points of the piles to make the pile piers as stiff as possible (see Plate LXXXIV).

After the trusses were in place the second time, the timber was pretty well dried out and there was no shrinking or sagging from the bearings first taken.

The timber girders which rested upon the top chords of the trusses were framed from six pieces of 14 x 14 ins. x 20 ft. gum timber. They were bolted together and keyed. To get the timber girders in place a portion of the old pier had to be removed (see Plate LXXXIV). A good deal of raising and lowering of the fixed spans was necessary, which was done by hydraulic jacks.

There were four pile piers, 12 piles in each pier, or 48 piles to support the two trusses. The piles for the deep water were ordered 68 to 70 ft. long and were 10 to 12 ins. in diameter at the small end. They were of oak and gum timber. In driving the piles the points were spread apart and the tops pulled into place under the caps. This was done to make the pile pier as stiff and stable as possible to withstand the force from barges and boats running into them. Sway bracing was bolted on by a diver, as shown on Plate LXXXIV.

The Pneumatic Caissons.—There were two caissons. That for the pivot pier, the dimensions of which were 36 x 36 x 16 ft., surmounted by a timber crib 36 x 36 x 25 ft., filled with concrete; and the smaller

caisson for the new draw rest pier 7, the dimensions of which were 16 ft. 9 ins. x 40 ft. 6 ins. x 12 ft., surmounted by a timber crib 16 ft. 9 ins. x 40 ft. 6 ins. x 29 ft., filled with concrete. The distance in each pier from the cutting edge of the caisson to the top of the crib or the bottom of the masonry pier was 41 ft. In each caisson and crib its horizontal section was the same throughout. The timber used was of white oak, excepting a few long pieces used in the working chamber for ties and struts and in the course built in the decking. These were of gum, because of the difficulty of getting oak sawed in sticks longer than 30 ft.

The design of these caissons is not particularly new or sufficiently different from other familiar forms to warrant a description here. Had the caissons for these piers been sunk in any open river, the work would have been interesting probably only from a commercial, not an engineering, point of view.

The old piers 7 and 9 were removed and the two new piers were built without interfering with the traffic of the road in any way.

The caissons for both of the piers were built upon the shore above the bridge, launched and floated into position at the pier sites when needed. The timber from which they were built, being of green oak, made it necessary to build a false bottom in each caisson, for neither could have been made to float without the use of some artificial means.

The position in which the pivot pier caisson was built is shown on Plate LXXXV. The plate is given to show into what shallow water a heavy caisson may be launched.

The launching ways were built well out into the river when they were constructed, to get into as deep water as possible and not have the grade of the ways too flat. The friction planks were extended beyond the ends of the heavy skidding timbers to change the grade and prevent the caisson from plunging into the mud and gravel and sticking fast as it ran off of the ends of the skid timbers.

When the work of building the caisson began, in June, the water was deep enough, but by the time we were prepared to launch the caisson there was barely enough water to float it at the end of the launching ways. The weight of the caisson ready to launch was 552 000 lbs. This includes the air lock and lower sections of shafting, cutting edge, and other iron bolts and drift bolts. The line of

PLATE LXXXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 713.
GAHAGAN ON JOHNSONVILLE BRIDGE.

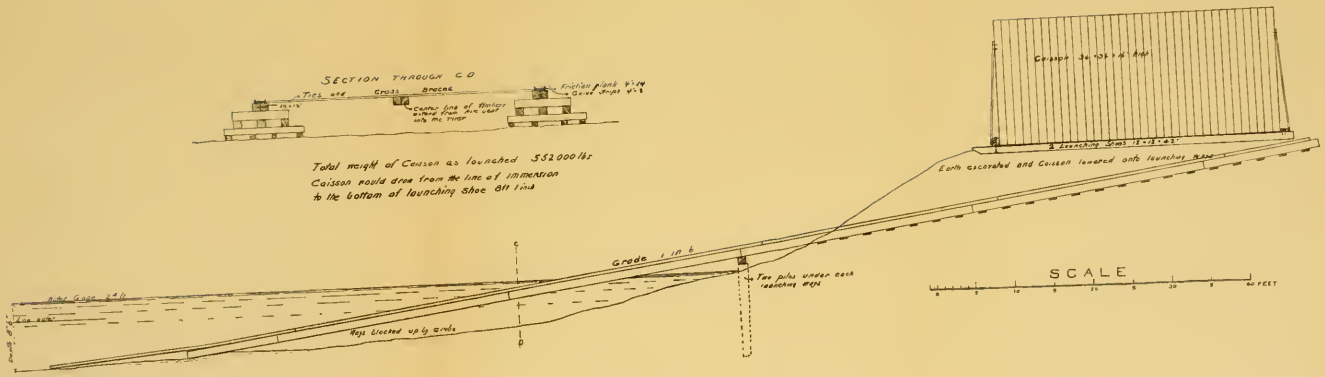
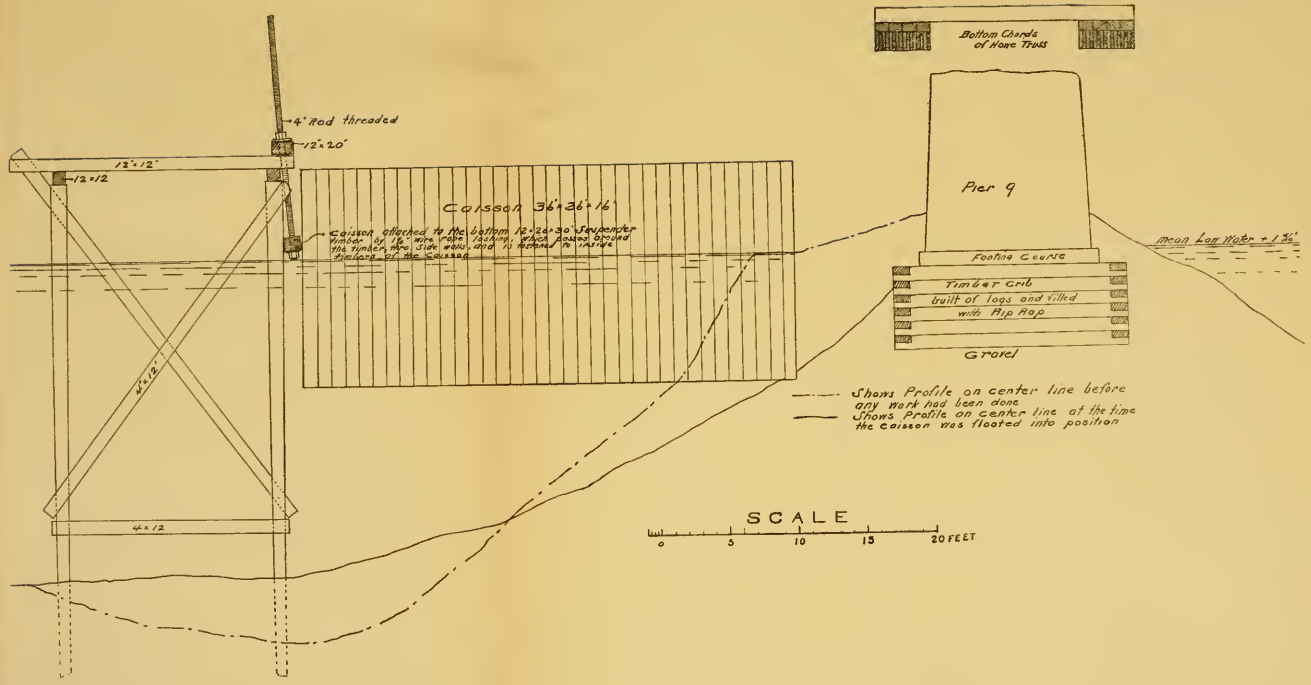


PLATE LXXXVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXI, No. 713.
GAHAGAN ON JOHNSONVILLE BRIDGE.



immersion was calculated as 8 ft. 1 in. above the bottom of the launching shoe, which was fastened to the caisson. The depth of water into which the caisson was launched was 8 ft. 6 ins.

Only two of the eight courses of decking were put in place before the launching. The balance were built in after the caisson was in place at the pier site. The false bottom for the pivot pier caisson was not caulked. It was made from 2 x 12-in. poplar; under this was put 1 x 5½-in. dry poplar tongued and grooved stuff. The joints were painted as this matched stuff was put on. Around the outside of the false bottom, next the cutting edge, a roll of oakum was laid in between the matched stuff and the 2 x 12-in. poplar. The false bottom leaked a good deal at first. It answered the purpose, however, and was built very quickly and cheaply.

After the caisson was launched, it was immediately floated down near the bridge into water about 10 ft. deep; we permitted it to fill, and let it settle on the bottom and remain there a few days. When we were ready to float the caisson into position on the pier site, the bottom had swelled, so there were few leaks, and the working chamber was easily pumped out with a Worthington pump of 4-in. suction. After the water had been pumped out, a hand pump would keep it down.

On Plate LXXXVI in the dotted line is shown the profile of the site for the pivot pier before any work had been done (the same profile is shown on Plate LXXXIII on a smaller scale). After the two trusses were in place at pier 9, the excavation was made, which changed the profile to that shown in the full line. This excavation was made to enable us to float the caisson into position; also to level up the pier site as much as possible. The site for the caisson was still 14 ft. lower on one side than upon the other. This side of the caisson, which could not take a bearing upon the ground, was supported from six piles. These piles were spaced 16 ft. center to center in one direction, and 14 ft. and 28 ft. center to center in the other. These piles were sway-braced to the ground. The braces were bolted on by a diver. This method of supporting the caisson is not new. It is very simple, and is shown on Plate LXXXVI.

Great care was used to keep the weight upon the two 4-in. screws as light as possible until the cutting edge had a bearing over two-thirds of its surface. The material upon which the caisson landed was principally rip-rap. This was removed from the high side next

the old pier, and deposited on the low side opposite. A great deal of this rip-rap was thrown out from under the cutting edge to build up the low side.

In placing the concrete in the crib, the side which hung on the screws was kept as light as possible. The concrete was put in only fast enough to keep it above water. On the side next the old pier which had a bearing upon the ground, the concrete was built up much faster. The leveling up of the bottom consumed five days. After the cutting edge was given a bearing all around, which, as has already been stated, was obtained by excavating on one side and filling up on the opposite side, the concrete was built up in the crib as rapidly as possible to give weight. This weight was supposed to be heavy enough to prevent the bank of material, and the weight from the lower portion of old pier 9, from crowding the caisson out of position.

The framing for this caisson was begun.....June 19th.
The caisson was launched.....Aug. 13th.
The air compressors were started.....Aug. 16th.
The caisson was landed on bed rock.....Sept. 16th.
Finished sealing the working chamber.....Sept. 20th.
Masonry for the pivot pier finished.....Oct. 14th.

In sinking the caisson there were piles which stood 7 ft. from the face of it which were not disturbed. These piles were driven about 15 ft. into the gravel and stood at the up-stream end of old pier 9. This leads the writer to believe that this caisson could have been sunk by the side of the old pier without disturbing it. The method which was used, however, was perfectly safe and did not take any chances of the settlement of the track and interfering with the traffic of the road.

The new draw rest pier No. 7, which was the last to be constructed, was made a very simple piece of work. The two trusses were removed from pier 9 and erected here. Old pier 7 was then torn down and the pier site excavated to a level of 8 ft. below low water. This practically removed all obstructions and left a comparatively level site for the caisson to be landed upon. The sinking was quickly and easily done.

The most difficult work connected with this pier was the masonry, which consumed the most time. Much of this work was done at night because the receiving of materials for it interfered with the superstructure workmen who were raising the grade of the track 4 ft. 6 ins. and

erecting the steel for the new draw span. It was slow work getting the heavy stone on to the pier because of the small amount of room between the trusses for such work.

The time consumed in building this pier is as follows:

Began framing caisson	Sept. 14th.
Finished building	Oct. 8th.
Launched caisson	Oct. 30th.
Started air compressors	Nov. 3d.
Landed on bed rock	Nov. 18th.
Finished sealing working chamber	Nov. 20th.
Finished masonry	Dec. 16th.

The material through which the caissons were sunk was gravel. The caissons rest on bed rock. The cutting edge in the pivot pier caisson is 44 ft. below low water, and that of the small caisson is 44.08 ft. below low water.

The concrete in the caisson and cribs was mixed in the proportions of 1 part of cement, 2 parts of sand, and 5 parts of broken stone. The stone used in the masonry was obtained from the Newsome quarry, which is at Newsome station, 16 miles west of Nashville, on the Nashville, Cincinnati and St. Louis Railway. Masonry backing was used in the piers. The mortar used throughout was made from Louisville cement, mixed in the proportion of 1 part cement and 2 parts sand. The cement was all tested before being used.

The quantities, as measured in the finished piers, and their cost, are as follows:

Pivot Pier.

119 792.4 ft. B. M., timber in caisson, at	$38 00	$4 552 11
95 727.5 ft. B. M., timber in crib, at	28 00	2 680 37
54 975 lbs. iron in caisson and crib, at	04	2 199 00
96 linear feet shafting, at	7 00	672 00
44 linear feet sinking below low water, at	344 83	15 172 42
313.4 cu. yds. material locked out, at	35 00	10 969 00
1 085.9 cu. yds. concrete in crib and caisson, at	6 00	6 515 40
233.5 cu. yds. concrete in air chamber, at	12 00	2 802 00
Total for caisson and crib		$45 562 30
2 112 cu. yds. displaced by caisson and crib, at	21 57	$45 562 30

Masonry of Pivot Pier.

415.62 cu. yds. face stone, at	$12 00	$4 987 44
725.19 " backing, at	7 00	5 076 33
24.52 " coping, at	16 00	392 37
Total cost of masonry		$10 456 09
1 165.33 cu. yds. masonry, average cost	8 97+	$10 456 09
Total cost of pivot pier		$56 018 39

Draw Rest Pier No. 7.

54 950 ft. B. M., timber in caisson, at	38 00	$2 088 10
60 097 ft. B. M., timber in crib, at	28 00	1 682 72
40 964 lbs. iron in caisson and crib, at	04	1 638 56
108 linear feet shafting, at	7 00	756 00
44.08 linear feet sinking below low water, at.	217 01+	9 565 87
17 cu. yds. material locked out, at	35 00	595 09
564.2 cu. yds. concrete in crib and pockets, at	6 00	3 385 20
108 cu. yds. concrete in air chamber, at	12 00	1 296 00
Total for caisson and crib		$21 007 45
1 107.51 cu. yds. displaced by caisson and crib, at	18 96	$21 007 45

Masonry of Pier No. 7.

351.39 cu. yds. face stone, at	12 00	$4 216 68
124.72 " backing, at	7 00	873 04
15.78 " coping, at	16 00	252 48
Total cost for masonry		$5 342 20
491.89 cu. yds. masonry, average cost	10 86	
Total cost of draw rest pier		$26 349 65

DISCUSSION.

H. G. Kelley, M. Am. Soc. C. E. (by letter).—The paper presented by Mr. Gahagan relates to a class of work which has been increasing within the last few years, and is due to the rapid enlargement in the volume of railroad traffic, necessitating heavier motive power and consequent wearing effect upon structures built for lighter service, and during a period when the financial necessities of railroad enterprises precluded large expenditures.

The responsibilities resting upon the engineers in charge of such structures, and the grave difficulties encountered in replacing or strengthening them, can hardly be appreciated by those whose experience and practice has been entirely with new work, but its importance merits careful attention and ample discussion from the members of this Society.

The writer of this discussion recently presented to the Society a paper upon a piece of work of this class, entitled "The Removal of a Defective Pivot Pier and its Reconstruction." He was also called upon in 1890 to perform another piece of work of the same kind, of even more importance, although not more difficult than those above mentioned. He can therefore appreciate the difficulties encountered by Mr. Gahagan, and the skill with which they were overcome, and desires to call attention to some of the principles governing work of this class, the dangers to be apprehended, and, incidentally, to present a history of a piece of work completed in the Arkansas River, which developed some valuable knowledge.

Governing Principles and Conditions.

First.—The traffic of the road must be continuously provided for without interruption.

Second.—If the stream is navigable, the traffic of the river must not be interrupted or stopped, for all such interruptions are followed by claims and suits for damages.

Third.—The freedom of the work is contracted by the superstructure overhead, limiting the space and preventing the convenient location and use of hoisting engines and derricks.

Fourth.—If the original piers were not founded upon bed rock, hardpan, or a strata of gravel upon which the new ones are also designed to rest, settlement of the original structure can be expected, with all the attendant dangers and expenses, and also liability of collision with the new pier.

Fifth.—The temporary contraction of the water-way increases the scour of the river bed, and may endanger the stability of the original piers.

Each case requires different treatment and a thorough study of the

surrounding conditions, as well as an intimate knowledge of the action of different soils under pressure with the influence of water.

By reference to Plate LXXXIII of Mr. Gahagan's article, which is drawn to natural scale, both vertical and horizontal, it will be seen that the scour had been exceedingly heavy, leaving the old piers upon a very insecure footing. Had the river bottom been other material than gravel, they could not have stood. Their insecurity is shown by the paragraph on pages 589 and 590, where it is stated that upon an attempt to drive piling within a few feet of pier 9, a settlement of 4 ins. occurred, throwing the pier out of position to the west, indicating the danger to be apprehended by disturbing the foundation material when the caisson was being sunk. The use of the auxiliary truss removed this danger and advanced the work to the second stage, where the caisson was placed in position ready for sinking.

Referring to that paragraph on page 590, mentioning the sagging of the auxiliary truss, it is not quite clear why the shrinkage could not be taken up, whether it was from the lack of sufficient thread on the tie rods, or on account of the weight supported by the truss; if the former was the reason, the difficulty could have been remedied by slacking away on one rod at a time, and putting a thick oak block underneath the nut on the top chord, and adjusting the entire span after all rods had been blocked; the writer has done this upon Howe truss spans of 125 ft. without difficulty.

The delicacy required in handling a pivot caisson suspended at one end and resting on a gravel bank at the other will be appreciated by those experienced in pneumatic work; but it is questionable if there would have been as little pressure from the bank tending to throw the caisson out of position, as was indicated, had it not been for the fact that the old pier 9 had been taken down and the load from the superstructure was carried by the auxiliary span.

Referring now to a similar piece of work in the Arkansas River, completed in 1891, a short history of the road and the conditions leading up to the necessity of the work will render the description more clear. In the year 1883, the Texas and St. Louis Railway was built from Waco, Tex., to Bird's Point, Mo., opposite Cairo, Ill., a total distance of 720 miles. This railroad was built upon the narrow-gauge system, the gauge being 3 ft., and all the bridges were built for light-weight engines and cars.

In 1887, the road changed ownership, and was widened to the United States standard gauge of 4 ft. 8½ ins. Heavier engines and cars then being constructed, it became necessary to remove all defective or inadequate structures and replace them with structures calculated to carry modern engines and trains. The writer personally assumed charge of this work in 1888.

Among the principal streams crossed was the Arkansas River, which was spanned by the following structures: Two spans of 100 ft. each,

combination wood and iron; five spans of 200 ft. each, combination wood and iron; one draw or swing span of 355 ft., iron.

These fixed spans were supported upon wrought-iron cylinders 7 ft. in diameter, filled with concrete. All of the combination spans were replaced in 1888 upon the original piers, but the draw span was carried until 1890. At this time the management desired to still further increase the weight of the engines used, and a thorough examination of the draw span and pivot pier resulted in a condemnation of both, the pier having already shown a movement to the north and signs of failure, this movement having been sufficient to block the opening of the draw, and requiring the removal and adjustment of some of the timbers in the end of the 100-ft. combination span adjoining it. This pivot pier consisted of a central cylinder 6 ft. in diameter, surrounded by six cylinders, 4 ft. in diameter, each braced together by latticed channel irons and tension rods. These cylinders were recorded to have been sunk in their construction to a depth of 68 ft. below extreme low water, with the exception of two of the small cylinders which had been stopped at about 40 ft. penetration, but no record was left to show which of these cylinders they were. The Arkansas River is navigable at all seasons of the year, and the cost of damages sustained by the river boats during the time in which the channel might be closed was an important item of expense to be considered in comparing the different plans for the reconstruction. The river is also subject to extreme floods, making all work hazardous, and the maintenance of false work especially dangerous. It was considered impracticable to attempt to obstruct the channel with false work, take down the old spans, and then remove the pivot pier by sinking a pneumatic caisson immediately above; for, besides the cost of damages to the navigation interests, the delay and expense of breaking up the old cylinders and taking them out through the air-lock of the caisson would have been exceedingly great.

The Department of War of the United States Government had fixed 160 ft. as the least clear channel-way that would be allowed; the only way, therefore, to overcome the difficulty was to increase the total length of the new draw span to admit of a masonry pivot pier to be sunk by the pneumatic method far enough in front to clear the old pivot pier and be in line with the axis of the bridge.

The danger to be apprehended was that the old pivot pier would settle while the caisson was being sunk so close to it, and thus not only endanger the old draw span, but cause the two piers to come in collision, and thus cause the destruction of both.

Soundings in the river showed the bottom to be of fine sand for a depth of 103 ft. below extreme low water; calculations showed that with a caisson of 32 ft. square penetrating to a depth of 68 ft. below extreme low water and rising to within 5 ft. of extreme low water, upon which would rest a cylindrical masonry pier 28 ft. in diameter and

$35\frac{71}{100}$ ft. high, sufficient stability would be had to carry a draw span of 428 ft. in length, and of the design and weight necessary to carry the established train loads. These dimensions were, therefore, decided upon, and gave 36.5 ft. as the distance between the centers of the old and new pivot piers, or a clear distance between the wall of the caisson and the closest 4-ft. cylinder of but 5.5 ft. (see page 601). This design necessarily required the removal of the old pivot pier and pier 2, and also the building of a new pier "A," shown on the plate; but this latter work involving no new principle, it will not be mentioned.

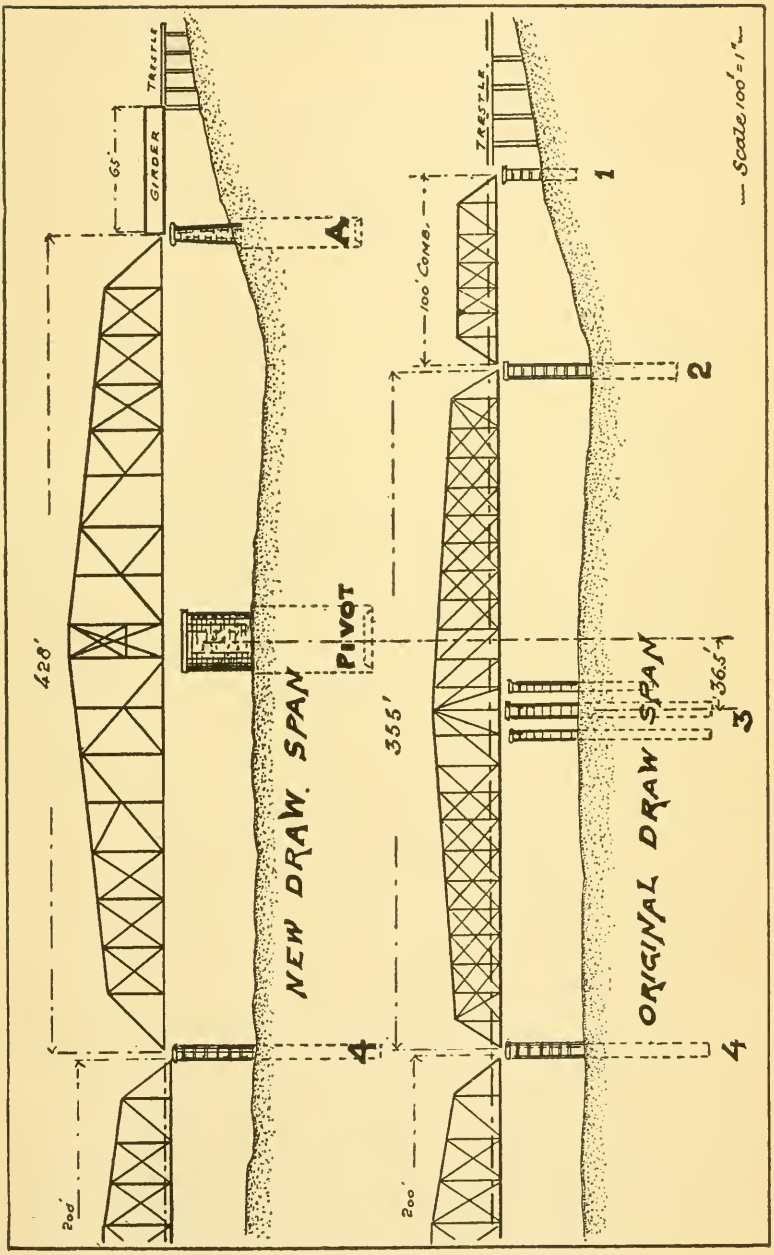
The work was let by contract, and Mr. J. A. L. Waddell, M. Am. Soc. C. E., prepared the caisson and masonry plans and also estimates for the contractor, which plans were approved by the writer. Mr. W. H. Gahagan, then Junior Am. Soc. C. E., was appointed Principal Assistant, and Mr. J. W. Schaub, M. Am. Soc. C. E., prepared the designs for the draw span.

The action of the caisson in descending so close to the existing structure was a matter for consideration; there were two possibilities:

First.—The proximity of the old pier might act like a high, sloping bank of earth and force the caisson out farther into the channel.

Second.—The river bed being composed of fine sand, it would be necessary to keep the caisson load heavier than usual and drop it suddenly and quickly at each settling, to prevent a disturbance in the surrounding sand and movement of the old pier. In this case the sand between the cylinders of the old pier would be obstructed in its flow and probably be more compact than the sand in the river bed; therefore, when the caisson was blown and the sand flowed into the working chamber as it settled, would not the sand coming from between the caisson wall and the old pier leave a partially open void before it could be filled from the surrounding river-bed material, and, the pressure from the channel side being greater, gradually force the caisson closer to the old pier ?

The actual result of the sinking proved the second possibility to be correct, but the contractor, naturally holding to the first, set his caisson 2 ft. closer to the old pier than was the correct position, with the result that it was only by the strongest efforts the caisson was kept from colliding with the old pier, and was finally landed 21 ins. closer to the old pier than was intended. The serious nature of the work was thoroughly appreciated and the strictest watch was maintained throughout; the caisson was handled cautiously, but the sinking was executed as rapidly as possible, and no serious delays or emergencies arose throughout the work. It was found, however, that when the caisson had reached a penetration of 40 ft. below extreme low water, the 4-ft. cylinders nearest the caisson showed a tendency to settle, indicating that these were the cylinders not originally sunk to the required depth. This settlement was slight, but was sufficient to throw the bracing of the cylinder cluster into severe tension, and


TRESTLE
GIRDER
65'
A
420'
200'
NEW DRAW SPAN
PIVOT
4
355'
100' CONS.
TRESTLE
1
2
3
36.5'
ORIGINAL DRAW SPAN
4
200'
Scale, 100' = 1"

threw the draw span northward about $\frac{1}{10}$ ft.; this, however, did not interfere with the passage of trains or the turning of the draw span to allow boats to pass through.

When the caisson reached its final penetration of 68 ft., the water in the river stood 20 ft. above low water, giving a total submersion of 88 ft.; the air pipe leading to the working chamber indicating a pressure of 37.5 lbs. per square inch, or the equivalent of 2.5 atmospheres above the normal atmospheric pressure. The men were worked in watches of four hours' duration, and but little trouble was had from "caisson disease."

The contraction of the channel produced a scour of 27 ft. between the new pivot pier and pier "A," which filled up at once when all obstructions were removed. After the new piers had been completed, false work was built in the river, the old spans removed and the new span erected. The old piers were then taken down to 7 ft. below extreme low water, and the river opened for traffic after a total closure of six weeks.

M. Ward Easby, Assoc. M. Am. Soc. C. E.—I would like to say, relative to some of the streams in western Pennsylvania, that I have had a little experience on the Conemaugh River at Johnstown. The Pennsylvania Railroad bridge is built down to rock. On February 17th, 1890, we had a little high water. After that high water we took soundings alongside of the pier, and found that the bottom of the river had been scoured out over 11 ft. I have also seen a bridge, the foundations of which I had the pleasure of tearing out, which was carried away by the flood in Johnstown in 1889, and which was located about 1 100 ft. below the Pennsylvania Railroad bridge, where two of the piers were undermined by the current. They probably had been put down about 7 ft. below the bed of the river. I built two bridges across that river, and I should recommend going down more than 5 or 6 ft. in that neighborhood if the bridges are to stay.

P. F. Brendlinger, M. Am. Soc. C. E.—This is certainly a very interesting paper, and I presume that every engineer, and contractor also, is interested in the removal of the old bridge and piers and the putting in of the new ones. There is one thing lacking in this paper, as there is in many papers. There is not sufficient history given; in fact, there is no history given whatever of the construction of this bridge. That, it seems to me, is a very important omission. It is always best to know the cause of such a construction. Here is a bridge, which is a very important one, built on, I think, as bad a substructure as I ever heard of or saw. It is a pity that the names of the persons who designed or constructed these foundations are not given. Of course, the author may have felt a little delicate about bringing up personal matters, but, at the same time, I think this should be known. The idea of building a pier without mortar in it seems to me perfectly absurd. The cribs are built of round logs and filled in with rough rip-rap stone. The author

does not say whether on top of that stone there was a grillage or timber platform or not; but, judging from his paper, there was not. The masonry was merely put on top of the crib, filled with rip-rap stone, and then built up. The designer may have had something in his mind as to what he would do with it after it was built dry, just as a certain engineer some time ago intended to build tunnel masonry of dry brickwork and then grout it, because we had grouted dry or bad brickwork in the New York aqueduct, with success. There we found dry brickwork and then injected grout to convert it into cemented brickwork; but the idea of designing a structure, putting it up dry and then pumping in grout afterwards, is something that I should think no sane engineer would do.

In looking at this plan, of course, the scale seems very much out of proportion, and it is a pity that it could not have been drawn to a natural scale. The holes that show the river bed to have been scoured out I presume would not look so badly on a natural scale. I do not understand why it is that there is such a tremendous scouring, unless there is a great current, of which there is no mention made at all. I know of but one bridge on the Ohio River (above Wheeling), or the Allegheny and Monongahela rivers that is not built on gravel. I think I built six bridges myself on the Ohio and Monongahela and Allegheny rivers; all but two were on solid timber foundation, resting on the gravel bed. The two exceptions had hollow cribs, sunk to place with stones and topped with a solid platform. The first one I built in 1872. The gravel has a depth of about 40 ft. The bridge is built about 500 ft. below Dam No. 2, Monongahela River, where the current is exceedingly swift. There is one span of 250 ft. and the other spans of 133 ft., about nine in all. The water was about 7 ft. deep, except at one corner of the dam, where it was scoured to 40 ft. We did not go to bed rock, but excavated about 7 ft.; one crib was built 14 ft. deep. But there is no dry masonry in those piers. As I say, that bridge was built about 22 years ago. I have never heard of any settlement or any scouring taking place. The piers were, of course, well rip-rapped.

On the bridges of the Vanderbilt system across the Monongahela River and the Youghiogheny River we built solid timber cribs on top of the gravel.

The Point Bridge at Pittsburgh that I built in 1875 has an 800-ft. span, piers 96 ft. high, in cement masonry resting on solid timber cribs built on gravel and well rip-rapped, and there is no sign of scouring.

The suspension bridge across the Allegheny River at Pittsburg, now torn down, built by Roebling 40 years ago, was built on timber cribs on gravel. They simply took a rake and raked the bottom clean and put down a timber foundation. Those rivers are exceedingly swift in high water and the water comes up very suddenly, and of course there is a very rapid current.

The only bridge in the Ohio River above Wheeling built on solid

rock is the bridge connecting the Pan Handle road with the Ft. Wayne road. I am therefore inclined to believe that if these piers had been built of good solid (cemented) masonry resting on a good crib built of 12 x 12-in. timbers, it would not have made much difference whether the middle of the river had been scoured out or not.

There is one thing here which I cannot quite understand. The author does not say much about it. How he erected that new draw span and at the same time worked the old pivot span and had to keep that draw in such a position that it could be moved for trains and boats. How he got the new draw in position on the new pivot pier, opening and closing the old every day, is something I don't understand.

W. H. Gahagan, Assoc. M. Am. Soc. C. E. (by letter).—Replying to Mr. Kelley's discussion of the settlement of the large trusses and why we did not take the sagging out of the temporary trusses by screwing up on the vertical rods, I would say that we did this very thing, and iron washer plates were put in over the gib plates and the nut put on again. This was done upon one rod at a time. We packed as much as 2½ to 3 ins. in this way, but were not able to do more than to hold and take up part of the shrinkage in the timbers. We could not lift the trusses with the rods after the dead load of the old spans was upon them.

All of this sagging could have been removed by blocking upon bents under the panel points (which had been removed); but it was not considered necessary to replace these bents.

By carefully examining the drawings and the dead load from the old spans to be carried, and the train load that was carried, it will be seen that these trusses needed to be very different from the ordinary type of Howe truss of the same span of which Mr. Kelley speaks.

In replying to Mr. Brendlinger, I would say that he is mistaken in regard to the scale of the drawings. They are all drawn to natural scale, the same horizontal as vertical, the scale being given upon each plate. By referring to Mr. Kelley's remarks, it will be noted that he calls attention to this fact in his discussion, and also notes the great amount of scour indicated between the piers. These scoured places between piers and below them are not exaggerated conditions, but are as shown on the plates. The old piers were setting upon gravel mounds, the contours of which are shown by examining plates LXXXIII and LXXXIV. The slopes or skin of these mounds were protected by rip-rap; and any one by examining these plates will see what would be likely to happen by disturbing the slopes or breaking the rip-rap skin upon them.

Nearly all of the piers of the bridge show a settlement on the down-stream end, some of them as much as 6 ins.; and to me it looked as though a very little disturbance of the down-stream slopes of old piers 7 and 9 might cause those slopes to slide at any time, and the pier to settle 1 ft. or more down stream, if the pier did not go to pieces entirely.

With regard to the history of the construction of the old bridge I

am not familiar, except that it was built 20 to 25 years ago, under conditions which certainly were very different from those existing to-day. Prices of material and work were many times what they are now; precedents and styles of construction were not so well fixed as they are to-day; and, possibly, this bridge could not have been built for lack of funds had an expensive type of construction been adopted.

The old foundation, as already stated, was built of logs framed into a crib. This crib was filled with rip-rap, and set on the gravel bed of the river, which was about 7 ft. below low water. The cut stone pile was placed upon this timber rip-rap crib. The piers as built had a batter of 1 in 12; and while the original constructors used no cement or mortar, the piers stood until they showed cracks from a settlement of the foundation which supported them.

In recent years, a later management thought to improve the condition of the piers by pouring cement grout into them. I examined these piers very closely while I was removing them, and they were not, and could not have been much improved by this grouting from the condition in which they were found. The grout in the first two or three courses at the top was good. It was hard and served as a rain cover to the portion of the pier below it. This portion of the grouting was valuable, but all below it might just as well have been left out, for it never set to become hard nor assisted to bind the pier together in any way. It was much like the tunnel Mr. Brendlinger spoke of, being grouted after being built.

The climate here is not as rugged as farther north, and masonry does not suffer much from frost.

The intention of the original constructor undoubtedly was to protect the foundation of these piers, as scour took place, by depositing rip-rap around them. This had been successfully followed, I understand, on all of the piers, until the Government objected to any more rip-rap being deposited around the draw rest and pivot piers, because it interfered with the passage of boats through the draw.

I cannot agree with Mr. Brendlinger in criticizing the style of design of this old work. It was built cheaply, and at an early day, when the traffic of the road was undoubtedly very light. It has, on the whole, done good service until it has been worn out, and the cause of the dangerous condition of the piers was not so much the fault of the designer as because the Government prevented the carrying out of the original method of protecting the foundations.

The current of the river during low water season is from 1½ to 2 miles per hour. The range between low and high water is about 49 ft. The speed of the current, I understand, at high water, varies in this portion of the river, *i. e.*, when the Ohio is low, and a heavy rise comes from the upper Tennessee. The current is very swift for this river—about 4 miles per hour. When the water in the Ohio is high, it has the effect of damming up the water in the Tennessee River

to some extent, and making a sluggish current at a very high stage of the river.

With regard to this old style of construction, I would say there is now, under my observation, in Idaho, a bridge 1 000 ft. long, of 10 Howe truss spans, crossing the Snake River. The current of the river is now 6½ miles per hour, and it is still more rapid at a higher stage of water. These bridges carry a heavy engine, which, with tender, weighs 195 000 lbs.

The foundations and piers for these bridges are framed timber cribs, which rest upon the bed of the river, and extend to the bottom chords of the trusses. These cribs are about 8 ft. wide, 30 ft. long, and about 25 to 30 ft. high.

They are filled with lava rock, which is abundant in the near vicinity. The rock is in pieces about the size one man can handle. The cribs are protected by rip-rap of this lava rock. The bottom of the river, between the piers, has scoured out as much as 20 ft. in some places since the piers were put in.

These bridges were built in 1883, at a time and place when it is said it required as large a force to provide for the commissary as to do the work. I am of the opinion that this and the style of foundation used by the Johnsonville builders was as good a construction as could have been adopted. The bridges have done the work required of them for their lifetime, and are now worn out, and it is doubtful if a more permanent style of construction for the foundations built at that time would not now have to be abandoned in putting in new superstructure, both from instability of the piers, and a rearrangement of the lengths of spans.

For these reasons, I think the original constructor of the Johnsonville bridge showed a good deal of skill in securing a structure which could be cheaply built, and which would carry the traffic of the road until there was need for something else.

Mr. Brendlinger says : "I am, therefore, inclined to believe that if these Johnsonville piers had been built of good, solid masonry, resting on a good crib, built of 12 x 12-in. timbers, it would not have made much difference whether the middle between the piers had been scoured out or not."

In some respects, I quite agree with Mr. Brendlinger, *i. e.*, the original spans should have been long enough not to materially reduce the section of the river, as has been done ; and the bottoms of the 12 x 12-in. timber cribs, he speaks of, should have been placed (by dredging the pier site) low enough to anticipate any scour from contraction of the channel, say, to a depth of 8 or 10 ft. below the old river bed. Had about 200-ft. spans been used, these old foundations could have been held, I think, by the method used by Mr. Brendlinger.

I had nothing to do with the erection of the iron work, but in answer to the question would say, that during the erection of the new draw, boats did not pass the bridge for six weeks.

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